Twinning Project MD 13 ENPI OT 01 16 (MD/26)
Support to promote cultural heritage in the Republic of Moldova
through its preservation and protection

Component 3

Quality Vocational Education and Training (VET) programs related to the protection and restoration of cultural heritage at secondary vocational education and at Higher Education (HE) levels developed and implemented

---

Activity 3.5

Creation at least four pilot projects for the practical training of VET and HE students (on-the-spot/on-the-job training) in the field of protection and restoration of cultural heritage. Input of private sector

REVIEW AND ANALYSIS OF THE SCIENTIFIC PRINCIPLES, STANDARDS AND REQUIREMENTS FOR THE REHABILITATION OF MASONRY HERITAGE BUILDINGS

DRAFT

Table of content

1. Executive summary and road map 4

2. Structural rehabilitation: theories, paradigms and Charts 7

3. Standards and codes 15

3.1 Eurocodes and US codes 15

3.2 ASCE/SEI 41-06 (Seismic Rehabilitation of Existing Buildings) 15
3.2.1 Target building performance levels 16
3.2.2 Rehabilitation objectives 19
3.2.3 Acceptance criteria for evaluation of performance levels 20

3.3 Eurocode 8 (Design of structures for earthquake resistance) 21
3.3.1 Part 1: General rules, seismic actions and rules for buildings 21
3.3.2 Part 3: Assessment and retrofitting of buildings 22

3.4 Italian codes 23
3.4.1 General framework 24
3.4.2 Instructions for monumental buildings 26

3.5 Moldovan codes 28
3.5.1 General framework 28
3.5.2 Moldovan laws and codes in context of existing buildings 29
3.5.3 Moldovan laws and codes in context of heritage buildings 30

4. Methods of modelling and analysis of historic masonry structures 32

4.1 Modelling issues related to heritage buildings 32

4.2 Types of damage and classes of buildings 37
4.2.1 Class A - Damage to in-plane loaded walls 39
4.2.2 Class B - Damage to out-of-plane loaded walls 44
4.2.3 Class C - Damage to masonry elements under combined axial and bending loads 50
4.2.4 Class D - Damage to in-plane loaded arches (or vaults) 52
4.2.5 Class E - Local damage of masonry 55
4.2.6 Class F - Rocking of single or multiple blocks 57
4.2.7 Class G - Unthreading or disconnection of structural elements of roofs and floors 58
4.2.8 Class H - Drift of vaults or floors in their horizontal plane 59
4.2.9 Class I - Damage to domes 60
4.2.10 Classification of architectonic assets 61
4.2.11 Macroelements 76
4.2.12 Correlations among architectonic assets, macroelements and damage 83

4.3 Review of models for masonry constructions 87
4.3.1 Classes of models 87
4.3.2 Continuum constitutive laws 89

DRAFT

4.3.3 Structural elements models 91
4.3.4 Interface models 97
4.3.5 Macro-blocks models 100

4.4 Distinctive features and tricks related to the different types of models 105
4.4.1 Modelling by structural elements models (SEM) 105
4.4.1.1 Identification of piers and spandrels 108
4.4.1.2 Modelling of structural elements 110
4.4.1.3 3D assembling of masonry walls 116
4.4.1.4 Modelling of diaphragms 117
4.4.1.5 Modelling of foundations and soil 120
4.4.2 Modelling by macro-block models (MBM) 121
4.4.2.1 Identification of the collapse mechanism 125
4.4.2.2 Modelling of limited compressive strength 125
4.4.2.3 Modelling of connections 127
4.4.3 Modelling by Continuum Constitutive Laws Models (CCLM) 130
4.4.3.1 Choice of the elements and mesh size 130
4.4.3.2 Modelling of walls and vaults 132
4.4.3.3 Modelling of diaphgrams 134
4.4.3.4 Required geometrical and mechanical data 135
4.4.4 Modelling by Discrete Interface Models (DIM) 135
4.4.4.1 Main features and application 135
4.4.4.2 Required geometrical and mechanical data 137

4.5 Modelling strategies for the different classes of architectonic assets 137
4.5.1 Class A (Palaces) 140
4.5.1.1 Issues on the modelling of the local mechanisms for class A 141
4.5.1.2 Issues on buildings in aggregate or without a clear box-behaviour 143
4.5.2 Class B (churches and religious buildings) 144
4.5.2.1 Consideration on the modelling of the macroelements 145
4.5.2.2 Consideration on the 3D CCLM linear model 147
4.5.3 Class C (towers) 152
4.5.3.1 Consideration on the modelling of towers (C2) 153
4.5.3.2 Issues on the modelling of the local mechanisms the bell tower (C2) 156
4.5.4 Class D (Triumphal arches, Vaults) 158
4.5.4.1 Considerations on modelling 2D mechanisms 158
4.5.4.2 Considerations on modelling 3D mechanisms 159
4.5.5 Class E (Massive structures) 161
4.5.6 Class F (Dry blocks structures) 162
4.5.6.1 Single Block dynamic response 163
4.5.6.2 Multi Blocks systems 163

4.6 Use of the models for the seismic assessment of heritage buildings 163

5. Interventions techniques 169

5.1 Strategies for the selection of interventions 169

5.2 Interventions to improve connections 170

5.3 Interventions to reduce thrusts of masonry vaults and their strengthening 172

DRAFT

5.4 Interventions to reduce the flexibility of horizontal diaphragms 173
5.5 Interventions on the roof covering structure.. 174
5.6 Interventions for increasing the strength of masonry panels.. 174

REFERENCES 178

Acknowledgements 190

The structure and the content of the present document has been elaborated by Chiara Calderini, Serena Cattari, Sergio Lagomarsino, Stefano Podestà Department of Civil, Chemical and Environmental Engineering, University of Genoa, Italy.

DISCLAIMER: This document was produced with the financial support of the European Union. Its contents are the sole responsibility of the authors and of the Italian Twinning Team and do not necessarily reflect the views of the European Union.

DRAFT

1. Executive summary and road map

This report contains a state of the art on the structural rehabilitation approaches, as well as their practical application in normative documents.

The main aim of the document is to provide an analysis of the approaches currently adopted in Moldova, to compare them with the European ones and to define a road map to improve and align them to European good practice and thereby achieve more effectiveness in ensuring both safety of historic buildings and the preservation of their heritage features.

In Chapter 2, a discussion of the main structural rehabilitation issues is proposed and the shared European rehabilitation principles are reported and commented. In Chapter 3, a review and a discussion on the main international and Moldovan normative documents on structural assessment and rehabilitation of existing buildings is proposed. Chapter 4 contains a review of methods for the modelling and analysis of historic masonry structures, useful for the structural assessment of existing buildings. Chapter 5 describes rehabilitation techniques

The following main ideas have guided the authors of this document:

- the structural rehabilitation of buildings is a multi-disciplinary work where both safety and conservation issues should be taken in account;
- the structure of a building has a value and deserves to be conserved;
- the structural rehabilitation of heritage buildings should be always based on a quantitative assessment of their structural safety;
- better models and assessment models increase the safety and reduce the interventions;
- the safety level of heritage buildings should not necessarily be the same of new buildings;
- the deep knowledge of the building and of its structural behaviour is the first step toward conservation.

Given these principles, in the following the main findings of this research are reported.

On the basis of the analysis of Moldovan context, it can be stated that the following issues are particularly critical:

- lack of a clear and consistent normative framework in the general field of the constructions (extensive number of codes and technical norms, adoption of norms deriving from different and non-homogeneous cultural contexts, …);
- lack of norms specifically targeted to existing buildings in general, and to heritage building in particular;
- scarce integration between safety and conservation issues, leading to a scarce interaction between structural norms and architectural conservation codes and between the “Ministry of Economy and Infrastructure” and the “Ministry of Education, Culture and Research”;
- preference for reconstruction rather than for building rehabilitation and conservation;

DRAFT

- merely qualitative approach to structural rehabilitation, when chosen.

An inconsistent approach to the conservation of buildings, the lack of normative documents, and the preference for reconstruction rather than for rehabilitation, leads to the loss of many heritage buildings and inhibits the advancements in structural assessment methods (for technicians and stakeholders), building rehabilitation techniques and practices (for professionals, construction companies, products, etc…), and knowledge of the technical features of Moldovan heritage buildings and on their transformations over time. Codes and normative documents call for research, knowledge, experimental actions, new products related to the actual context of a nation. It is worth noting, in fact, that even if general principles are valid in any country, their application in a specific national context requires a deep understanding of its characteristics, in terms of cultural traditions, economic and production context, social environment and so on.

For this reason, the broadening of knowledge on the Moldovan context is the first step toward the revision and the improvement of Moldovan rehabilitation strategies. This step can be done with the support of Moldovan universities and research centres. In particular, the following issues should be faced:

- knowledge of the buildings: structural and architectonical features of Moldovan heritage buildings, in terms of overall organization, technical details, materials, etc…should be known in depth;
- knowledge of the building context: analysis of size, type and skills of construction companies, skills of workforce, scientific and professional preparation of architects and engineers, as well as products and practices, is very important to target assessment procedures and intervention techniques to the actual Moldovan context.

Thanks to these studies it will be possible to:

- define a structural assessment procedure specifically targeted on Moldovan heritage buildings;
- identify the most suitable strengthening interventions to be carry out for the improvement of Moldovan heritage buildings.

The second step should be a thorough revision of conservation approaches to heritage buildings in the Republic of Moldova, based on the recognition that the value of architectural heritage is not only in its appearance, but also in the integrity of all its components (including the structure) as a unique product of the specific building technology of its time and on a deep rethinking of the relationship between conservation and safety.

This step should be made by a close cooperation of universities and the Ministry of Education, Culture and Research.

The third step will be the draft of a normative document specifically targeted on the safety assessment and rehabilitation of Moldovan heritage buildings. It is very important that the contents of this document would be defined by a close cooperation of the "Ministry of Economy and Infrastructure" (in charge of safety issues), and the "Ministry of Education, Culture and Research" (in charge of conservation issues). The contents of this document should be aligned with both conservation codes and with technical norms for constructions (design of new buildings etc…). This normative document should contain the answers to the following questions:

DRAFT

- which is the objective of structural assessment and rehabilitation of heritage buildings in the Republic of Moldova?
- how the structural safety of a Moldovan heritage building can be assessed?
  (reference to the structural assessment procedure defined in the second step);
- what is the minimum safety level that the Republic of Moldova accepts for heritage buildings in order to conserve their historical value? is it the same of new buildings?
- what are the possible choices if the target safety level of a heritage building is not achieved?
- if the safety level of heritage buildings should be increased, which design approach and which techniques can be used?

The main problem for this step is that it would require, before its actual implementation, a more general revision of the overall Moldovan normative framework, leading to an integrated document on the design of new structures and the assessment of existing ones. The lack of integration between structural and conservation codes and between design and assessment norms is one of the main reasons of ineffectiveness of heritage building norms.

Finally, the fourth step is related to education and training. The lack of research and education in the field in heritage building rehabilitation is evident in the Republic of Moldova as in many European countries. However, it is well evident that no change can be made without research and without educate and training young generations in the field. Education and training should be made at different levels:

- at the university level, introducing specific courses on the structural rehabilitation of buildings and/or designing specific courses of study (at both bachelor and master level) expressively devoted to the conservation of heritage buildings;
- at the professional level, promoting specific training courses for already working professionals.

It is worth noting that this is a complex and long path that cannot be made but with a multi-disciplinary approach. The cooperation of different disciplines, the cooperation between historians, architects and engineers and the cooperation between different Ministries is the key for a successful and effective revolution in the field of the protection of heritage buildings.

DRAFT

1. Structural rehabilitation: theories, paradigms and Charters

It is worth noting that structural rehabilitation theories here presented are related to heritage buildings and not to generic existing buildings. This distinction is very important, since for generic existing building the safety problem is linked to economic/operative issues only, while for heritage buildings it is linked to conservation issues also.

Introducing structural rehabilitation theories requires a first analysis on the meaning of the word "rehabilitation" and on its many "synonyms" (strengthening, conservation, consolidation, …), often used improperly.

According to the Cambridge Dictionary, the word "rehabilitation" has the following meanings:

- *the process of returning to a healthy or good way of life, or the process of helping someone to do this after they have been in prison, been very ill, etc…*

- *the process of returning something to a good condition:*

This definition implies three considerations: somebody, or something, has suffered a damage; the damage was reparable; after the damage occurred, thank to rehabilitation, he/she/it will return to a good condition.

A first issue arises from this definition: how do we define the "good condition"? Is it the condition of the subject (person or thing) before the damage? Or is it an optimal reference condition? And what if the subject (person of thing) was not in an optimal condition even before the damage?

These issues are crucial in structural rehabilitation theories. Actually, one of the most crucial issue in structural rehabilitation theories is:

- what is the purpose of rehabilitation?

Many different answers were given over time. Here are the two main ones:

- the purpose is to return the building to its "initial condition";
- the purpose is to return the building to a "good condition", improving, if necessary, its structural behaviour;

(here the word "condition" implicitly means "structural safety" of the building).

In the first case, the problem is to define the "initial condition" of an historic building. This is not always an easy task, since heritage buildings were often subjected to many transformations, damage and reparations along their long life. This first approach raises a further critical question: what if the "initial condition" of the building was not "good"? Should we accept it?

In the second case, the problem is to define the reference "good condition" for an historic building. Let's consider the health/illness similitude.

A "good health condition" of a person is defined as his/her capacity to carry out ordinary activities and to deal with ordinary diseases. In the same way, a "good condition" of a structure implies its capability of carrying ordinary permanent loads and dealing with ordinary variable actions (wind, snow, frequent low intensity earthquakes, etc…). But what about extraordinary diseases and actions?

DRAFT

In the definition of the word "rehabilitation", it is implicitly assumed that the considered illness/damage is somehow curable/reparable. So which is level of severity of a disease that we assume to the curable? And which is the level of intensity of an extraordinary action producing a reparable damage? And what is a reparable damage?

It is well evident that the answer to these questions is not an easy task and may change over time, depending on our knowledge on actions, heritage structures and techniques of rehabilitation. As we are progressively improving our capability to cure serious infections and many type of cancer, in the same way we are improving our rehabilitation techniques, progressively moving the limit between reparable/unrepairable damage.

In this framework, going back to the aim of structural rehabilitation, it is a shared opinion that:

- the purpose is to return the building to a "good condition", improving, if necessary, its structural behaviour;
- the "good condition" of a building implies its capability of carrying ordinary loads and extraordinary loads producing repairable damage.

The debate is on the two previous questions:

- what is a reparable damage for a heritage structure?
- which is the level of intensity of an extraordinary action producing a reparable damage?

Let consider the first question. Since a heritage building is a cultural asset, its damage can be defined as repairable if it can be recovered without losing material and cultural integrity. But usually, a heritage building is also a functional asset and, thus, it should safe for people using it. Fortunately, damage reparability usually coincides with life safety. In short, therefore, a damage is considered repairable if it can be recovered without losing material and cultural integrity and if it does not produce casualties.

Let us now consider the second question. An extraordinary action can be defined as an action that is unusual, rare. From an engineering from of view, an event is rare if it has a low frequency of occurrence, or a low probability of occurrence. But how much low?

Dealing with building, seismic actions are the main extraordinary actions considered (for this reason, most of the theories on structural rehabilitation are related to seismic issues). But it is well known that the range of intensity levels of earthquake is wide and that intensity levels are strictly related to probability of occurrence: there are quite frequent low intensity earthquakes as well as very rare high intensity earthquakes. So the question becomes: which is probability of occurrence of an earthquake producing a repairable damage on a heritage structure?

Let assume that we are capable to calculate such probability of occurrence. How do we decide whether it is acceptable or not for our society? This is the main problem. The acceptability of the probability of occurrence of a hazard can be defined only in relative and considering the consequences of the hazard itself. It hard to say, without terms of comparison, whether a probability of occurrence of a hazard of 1% in 100 years is high or low. It depends on many economic, social environmental and cultural factors. Structural norms, mainly elaborated for new structures, have defined very clearly such acceptable probability thresholds. In the actual Italian structural code (NTC 2018), for an ordinary structure with a life span of 50 years at ultimate limit state, it is assumed that:

DRAFT

- standard variable actions (snow, wind, etc…) have a probability of occurrence/exceedance of approximately 63% in the life span of the structure;
- seismic actions have a probability of occurrence/exceedance of 5-10% (depending on the limit state considered, whether the life safety or the collapse) in the life span of the structure.

In practice, just to give an idea, the collapse of a new ordinary structure hit by an earthquake that has a probability of occurrence of 5% in 50 years is considered acceptable.

For better or for worse, new structures should be the term of comparison for heritage structures. Considering life safety only, it would be logical to consider the same probability of occurrence of variable and seismic actions for heritage and new buildings. But heritage buildings are usually less performant than new buildings. Following this approach, the risk would be to lose the material and cultural integrity of the cultural asset, since the damage produced by strengthening interventions would be even worse than the damage produced by earthquakes. So what? On the one hand, we should keep working on the improvement of strengthening interventions. On the other hand, we should accept lower safety levels for heritage buildings than for new buildings. This means, implicitly, that we should accept that the probability of occurrence of an earthquake leading a building to the life safety limit state should be higher for a heritage building than for a new building. But again: how much higher?

Up to now, nobody took the responsibility to answer to this hard question. For this and other reasons, most of structural norms do not consider heritage structures at all or consider them as special structures with special rules. This led to three different effects:

- on the one hand, even if not explicitly required by structural and seismic norms, most of engineers asked heritage buildings to have the same level of safety of new ones, designing in many case strong strengthening interventions;
- on the other hand, since not explicitly required by structural and seismic norms, many heritage buildings with very low safety levels were not strengthened at all, suffering in some cases severe damage.

Italian structural and seismic norms (that are the only ones that, since 1909, had explicitly and systematically addressed the issue of heritage buildings), for many years endorsed the principle of "case by case" (no rules). In most recent years (since 1996), the "case by case" principle was replaced by the "improvement" principle: engineers working on the rehabilitation of a heritage buildings were asked to generically improve their seismic response. But, again, in the norms the amount of improvement was not defined. This led, on the one hand, to many excessive strengthening interventions and, on the other hand, to many ineffective interventions.

Despite reaffirming the "improvement" principle, Italian Guidelines (2011) introduced a significant innovation: for the first time, engineers were asked to evaluate the amount of improvement obtained with their strengthening interventions. The novelty of this approach lies in the fact that, to evaluate the amount of improvement of their strengthening interventions, engineers was asked for the first time to calculate the safety level of heritage buildings (that is to calculate the probability of occurrence of seismic actions leading

DRAFT

a heritage structure to “reparable damage/life safety” limit state). The same approach was adopted in the very recent last version of the Technical Rules for Constructions (NTC 2018).

In this code, however, another relevant novelty was introduced: although stating that this rule may not be applied “*in specific situations related to heritage buildings*”, “improvement” interventions should lead to obtain a safety level (expressed in term of ratio between the seismic capacity and seismic demand) that is at least 60% of that of a new building. This means implicitly that, except for specific cases, Italian rules accept that the probability of occurrence of an earthquake leading a heritage building to the life safety limit state is higher than for a new building. In standard conditions, it can be approximately calculated that a reduction of 60% of the seismic demand corresponds to an increasing of the probability of occurrence of the seismic action by a factor of 3.5. In practice, except for specific cases, Italian rules accept that the probability of occurrence of an earthquake leading a heritage building to life safety limit state is approximately 3.5 times higher than for a new building: roughly, 35% against 10%. Although always avoidable thanks to the “specific cases” loophole, this is a bold choice.

it should be underlined that, in recent past, it was very difficult to calculate the probability of occurrence of seismic actions leading a heritage structure to “reparable damage/life safety” limit state (in the far past, the concept of probability of occurrence of an action itself was unknown). Even today, this is a very difficult (but not impossible) task.

This is one of the reasons why structural and seismic norms avoided to ask for evaluating the seismic safety of a heritage building for a long time.

This is also one of the main reasons why, in the past, engineers were induced to replace many old and hardly evaluable heritage structures with new and easily evaluable structures.

It has been stated that only in Italy the issue of heritage buildings was discussed and explicitly considered in structural and seismic norms. This is a credit. In the other European countries, this issue has always been stresses in other contexts, such as conservation charters and structural rehabilitation guidelines (see in particular ICOMOS Chart, 2003, ICOMOS Recommendations, 2003, ISO 13822, 2010, and CIB Recommendations, 2010). These documents do not attempt to answer to the awkward questions discussed in the first part of this chapter. The topics covered are more general, but not less relevant. The most advanced of this kind of documents is the ICOMOS Chart (2003). Here are reported its “Principles”, that are widely shared by European conservation institutions, researchers and practitioners:

1. General criteria

1.1 Conservation, reinforcement and restoration of architectural heritage requires a multidisciplinary approach.

1.2 Value and authenticity of architectural heritage cannot be based on fixed criteria because the respect due to all cultures also requires that its physical heritage be considered within the cultural context to which it belongs.

1.3 The value of architectural heritage is not only in its appearance, but also in the integrity of all its components as a unique product of the specific building technology of its time. In particular,

DRAFT

the removal of the inner structures maintaining only the façades does not fit the conservation criteria.

1.4 *When any change of use or function is proposed, all the conservation requirements and safety conditions have to be carefully taken into account.*

1.5 *Restoration of the structure in Architecture Heritage is not an end in itself but a means to an end, which is the building as a whole.*

1.6 *The peculiarity of heritage structures, with their complex history, requires the organisation of studies and proposals in precise steps that are similar to those used in medicine. Anamnesis, diagnosis, therapy and controls, corresponding respectively to the searches for significant data and information, individuation of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions. In order to achieve cost effectiveness and minimal impact on architectural heritage using funds available in a rational way; it is usually necessary that the study repeats these steps in an iterative process.*

1.7 *No action should be undertaken without having ascertained the achievable benefit and harm to the architectural heritage, except in cases where urgent safeguard measures are necessary to avoid the imminent collapse of the structures (e.g. after seismic damages); those urgent measures, however, should when possible avoid modifying the fabric in an irreversible way.*

2. Researches and diagnosis

2.1 *Usually a multidisciplinary team, to be determined in relation to the type and the scale of the problem, should work together from the first steps of a study - as in the initial survey of the site and the preparation of the investigation programme.*

2.2 *Data and information should first be processed approximately, to establish a more comprehensive plan of activities in proportion to the real problems of the structures.*

2.3 *A full understanding of the structural and material characteristics is required in conservation practice. Information is essential on the structure in its original and earlier states, on the techniques that were used in the construction, on the alterations and their effects, on the phenomena that have occurred, and, finally, on its present state.*

2.4 *In archaeological sites specific problems may be posed because structures have to be stabilised during excavation when knowledge is not yet complete. The structural responses to a “rediscovered” building may be completely different from those to an” exposed” building. Urgent site-structural-solutions, required to stabilise the structure as it is being excavated, should not compromise the complete building’s concept form and use.*

2.5 *Diagnosis is based on historical, qualitative and quantitative approaches; the qualitative approach being mainly based on direct observation of the structural damage and material decay as well as historical and archaeological research, and the quantitative approach mainly on material and structural tests, monitoring and structural analysis.*

DRAFT

2.6 *Before making a decision on structural intervention it is indispensable to determine first the causes of damage and decay, and then to evaluate the safety level of the structure.*

2.7 *The safety evaluation, which is the last step in the diagnosis, where the need for treatment measures is determined, should reconcile qualitative with quantitative analysis: direct observation, historical research, structural analysis and, if it is the case, experiments and tests.*

2.8 *Often the application of the same safety levels as in the design of new buildings requires excessive, if not impossible, measures. In these cases, specific analyses and appropriate considerations may justify different approaches to safety.*

2.9 *All aspects related to the acquired information, the diagnosis including the safety evaluation, and the decision to intervene should be described in an "EXPLANATORY REPORT".*

3. Remedial measures and controls

3.1 *Therapy should address root causes rather than symptoms.*

3.2 *The best therapy is preventive maintenance*

3.3 *Safety evaluation and an understanding of the significance of the structure should be the basis for conservation and reinforcement measures.*

3.4 *No actions should be undertaken without demonstrating that they are indispensable.*

3.5 *Each intervention should be in proportion to the safety objectives set, thus keeping intervention to the minimum to guarantee safety and durability with the least harm to heritage values.*

3.6 *The design of intervention should be based on a clear understanding of the kinds of actions that were the cause of the damage and decay as well as those that are take into account for the analysis of the structure after intervention; because the design will be dependent upon them.*

3.7 *The choice between "traditional" and "innovative" techniques should be weighed up on a case-by-case basis and preference given to those that are least invasive and most compatible with heritage values, bearing in mind safety and durability requirements.*

3.8 *At times the difficulty of evaluating the real safety levels and the possible benefits of interventions may suggest "an observational method", i.e. an incremental approach, starting from a minimum level of intervention, with the possible subsequent adoption of a series of supplementary or corrective measures.*

3.9 *Where possible, any measures adopted should be "reversible" so that they can be removed and replaced with more suitable measures when new knowledge is acquired. Where they are not completely reversible, interventions should not limit further interventions.*

3.10 *The characteristics of materials used in restoration work (in particular new materials) and their compatibility with existing materials should be fully established. This must include long-term impacts, so that undesirable side-effects are avoided.*

DRAFT

3.11 The distinguishing qualities of the structure and its environment, in their original or earlier states, should not be destroyed.

3.12 Each intervention should, as far as possible, respect the concept, techniques and historical value of the original or earlier states of the structure and leaves evidence that can be recognised in the future.

3.13 Intervention should be the result of an overall integrated plan that gives due weight to the different aspects of architecture, structure, installations and functionality.

3.14 The removal or alteration of any historic material or distinctive architectural features should be avoided whenever possible.

3.15 Deteriorated structures whenever possible should be repaired rather than replaced.

3.16 Imperfections and alterations, when they have become part of the history of the structure, should be maintained so far so they do not compromise the safety requirements.

3.17 Dismantling and reassembly should only be undertaken as an optional measure required by the very nature of the materials and structure when conservation by other means impossible, or harmful.

3.18 Provisional safeguard systems used during the intervention should show their purpose and function without creating any harm to heritage values.

3.19 Any proposal for intervention must be accompanied by a programme of control to be carried out, as far as possible, while the work is in progress.

3.20 Measures that are impossible to control during execution should not be allowed.

3.21 Checks and monitoring during and after the intervention should be carried out to ascertain the efficacy of the results.

3.22 All the activities of checking and monitoring should be documented and kept as part of the history of the structure.

ICOMOS Charters synthesises very well the consolidated European approach to structural rehabilitation. But it sets out an "ideal" path for structural interventions on heritage buildings that is far to be followed in real practice, own to many economical, practical and professional problems. The weakness of the Chart lays, once again, in the lack of clear responsibilities on the safety and conservation of ancient buildings and in considering the qualitative approach as an alternative to the quantitative one (based on the use of structural modeling), when it is evident that a combined use would be the optimal solution.

Following the path indicated by the ICOMOS Charter (2003), in 2010 two other documents were published on the structural assessment of existing structures: ISO 13822 (2010) and CIB 335 (2010). ISO 13822 (2010) contains an informative annex on heritage structures, which suggests to perform a preliminary assessment, based on investigations and historical analysis, and to adopt a detailed assessment only when the qualitative approach is not sufficient to obtain a clear picture of the building safety (diagnosis); therefore, numerical

DRAFT

modeling is required only in the latter case, and the unavoidable model deficiencies are supplemented by qualitative information (historical or comparative approach, engineering judgment). The assessment procedure proposed by CIB 335 (2010) coincides substantially with that of ISO 13822 (2010), but in addition some specific proposals on how to define safety coefficients and the confidence factor are introduced. Also in these cases, the qualitative approach is clearly predominant.

DRAFT

2. Standards and norms

2.1 Eurocodes and US norms

This section is devoted to the analysis of ASCE standard ASCE/SEI 41/06 and EC8, the two main European and American norms respectively. Both documents refer to earthquake risk and focus on performance-based approach. Although they do not contain specific directions on the application of the abovementioned approach to monumental buildings, a brief review of the approaches adopted for the definition of the performance levels and the earthquake hazard levels appears to be quite interesting.

In ASCE/SEI 41/06, the verification is based on the fulfilment of a set *Rehabilitation Objectives*, defined with reference to a combination of predefined *Target Building Performance Levels* and *Earthquake Hazard Levels*.

Target Building Performance Levels are referred to the achievement of specific damage levels, defined by proper combinations of effects on both structural and non-structural elements (performance level is a post-earthquake condition of a building, associated to a well-defined point on a scale measuring how much loss is caused by earthquake damage).

The *Hazard Levels* are associated with predefined probabilities of occurrence in a reference period or with predefined return periods. The quantitative evaluation of the performance levels at the scale of the structure is based on the fulfilment of a set of *Acceptance Criteria* defined at the scale of the structural elements (floor, columns, beams, etc.). Thus, provided that one structural element passes the acceptance criteria, the entire structure is considered to be at the correspondent damage level. The acceptance criteria are defined in terms of strength or displacement thresholds (drift, rotation) depending on the type of response of the structural element (whether fragile or ductile). Reference values and reliable models to evaluate such thresholds are defined for the different structural types. Moreover, as function of the accuracy of the model adopted for the description of the response of structural elements, more or less restrictive values are proposed. In § 3.2 a description of the different *Rehabilitation Objectives, Target Building Performance Levels and Acceptance Criteria* proposed in ASCE/SEI 41/06 is provided.

In EC8, two classes of requirements related to different degrees of reliability expected (basically referred to no-collapse and damage limitation conditions) are defined. Three limit states, corresponding to different levels of hazard, are associated with such requirements in the case of existing buildings. The hazard levels, defined in terms of probability of occurrence in a reference period or return periods, are modified through a coefficient depending on the "importance" of the building considered (in particular the "importance" is mainly related to the need that the structure remains operational after an earthquake). Concerning masonry buildings, differently from ASCE/SEI 41-06 Standard, the limit states are defined on the basis of the reaching of conventional limits defined on the overall capacity curve of the structure. In § 3.3 a detailed description of the limit states introduced in EC8 is provided.

2.2 ASCE/SEI 41-06 (Seismic Rehabilitation of Existing Buildings)

The provisions and commentary of this standard are based primarily on the ASCE/SEI 41/06, with limited material taken from the FEMA 274 and FEMA 356, was prepared by the American Society of Civil Engineers (ASCE) for the Federal Emergency Management Agency (FEMA), and it was developed from FEMA 273.

DRAFT

ASCE/SEI 41/06 supersedes FEMA 356, but FEMA 274 remains a valid explanation for its provisions. It is intended to be generally applicable to seismic rehabilitation of all types of buildings (regardless of importance, occupancy, historic status or other classifications of use). The seismic rehabilitation is defined as improving the seismic performance level of structural and non-structural components of a building by correcting deficiencies identified in a seismic evaluation, to prevent the building from achieving selected *Target Performance Levels*, defined by *Rehabilitation Objectives*.

2.2.1 *Target building performance levels*

A *Performance Level* is the intended post-earthquake condition of a building, a well-defined point on a scale measuring how much loss is caused by earthquake damage, while a *Performance Range* is a range or band of performance rather than a discrete level.

A *Target Building Performance Level* is a combination of a *Structural Performance Level* and a *Non-structural Performance Level*, both selected from a well-defined list of possible levels. These *Target Building Performance Levels* are discrete damage states, that have readily identifiable consequences associated with the post-earthquake disposition of the building, selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake.

The *Structural Performance Level* of a building is selected from four discrete *Structural Performance Levels* and two intermediate *Structural Performance Ranges* defined as follow:

- *Immediate Occupancy Structural Performance Level* (S-1). It is defined as the post-earthquake damage state in which a structure remains safe to occupy, essentially retains its pre-earthquake design strength and stiffness. The risk of life -threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate.

- *Damage Control Structural Performance Range* (S-2). It is defined as the continuous range of damage states between the Immediate Occupancy Structural Performance Level (S-I) and the Life Safety Structural Performance Level (S-3). It may be desirable to minimize repair time and operation interruption, as a partial means of protecting valuable equipment and contents or to preserve important historic features when the cost of design for immediate occupancy is excessive.

- *Life Safety Structural Performance Level* (S-3). It is defined as the post earthquake damage state in which a structure has damaged components but retains a margin against onset of partial or total collapse. Some structural elements and components are severely damaged but this has not resulted in large falling debris hazards, either inside or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical.

- *Limited Safety Structural Performance Range* (S-4). It is defined as the continuous range of damage states between the Life Safety Structural Performance Level (S-3) and the Collapse Prevention Structural Performance Level (S-5).

- *Collapse Prevention Structural Performance Level* (S-5). It is defined as the post-earthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin against collapse. Substantial damage to the structure has occurred, potentially including

DRAFT

significant degradation in the stiffness and strength of the lateral-force resisting system, large permanent lateral deformation of the structure, and degradation in vertical load carrying capacity (only in few elements, not significant in the gravity-load-resisting system). Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for preoccupancy, as aftershock activity could induce collapse.

- *Structural Performance Not Considered* (S-6). A building rehabilitation that does not address the performance of the structure shall be classified as S-6; for example, bracing parapets or anchoring hazardous materials storage containers without addressing the performance of the structure itself.

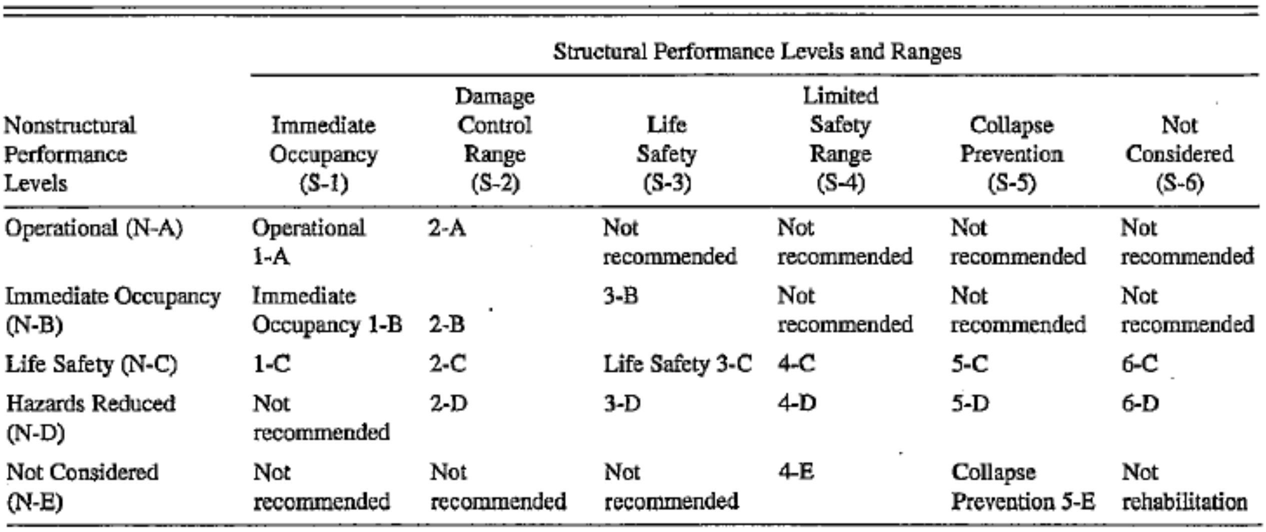
The *Non-structural Performance Levels* of a building shall be selected from five discrete levels:

- *Operational Non-structural Performance Level* (N-A). It is defined as the post earthquake damage state in which the non-structural components are able to support the pre-earthquake functions present in the building.
- *Immediate Occupancy Non-structural Performance Level* (N-B). It is defined as the post earthquake damage state in which non-structural components are damaged but building access and life safety systems including doors, stairways, elevators, emergency lighting, fire alarms, and fire suppression systems generally remain available and operable, provided that power is available.
- *Life Safety Non-structural Performance Level* (N-C). It is defined as the post earthquake damage state in which non-structural components are damaged but the damage is not life-threatening.
- *Hazards Reduced Non-structural Performance Level* (N-D). It is defined as the post earthquake damage state in which non-structural components are damaged and could potentially create falling hazards, but high-hazard non-structural components are secured to prevent falling into areas of public assembly. Preservation of egress, protection of fire suppression systems and similar life-safety issues are not addressed in this Non-structural Performance Level.
- *Non-structural Performance Not Considered* (N-E). A building rehabilitation that does not address non-structural components shall be classified as N-E.

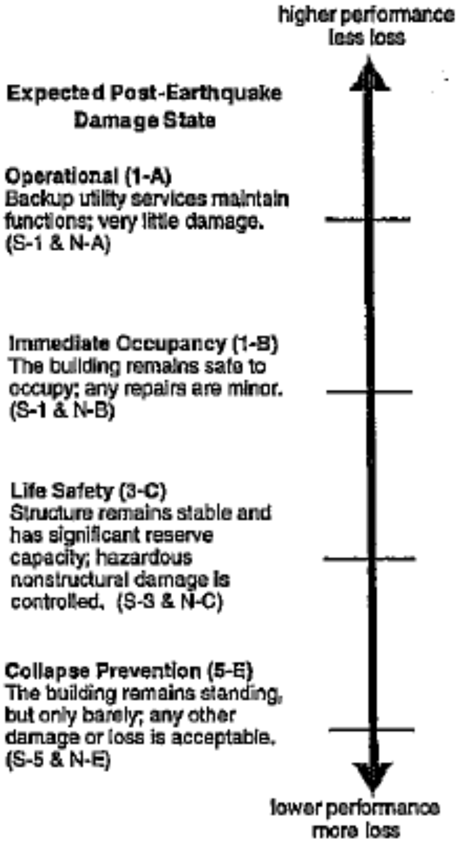
A *Building Performance Level* is designated alphanumerically with a numeral representing the *Structural Performance Level* and a letter representing the *Non-structural Performance Level* (such as 1-B or 3-C). Table 3.1 indicates the possible combinations and provides names for those that are most likely to be selected as a basis for design (Figure 3.1). Also Table 3.2 gives estimated descriptions of damage for these *Target Building Performance Levels*.

DRAFT

Table 3.1. Possible Building Performance Levels.



Nonstructural Performance Levels	Structural Performance Levels and Ranges Immediate Occupancy (S-1)	Damage Control Range (S-2)	Life Safety (S-3)	Limited Safety Range (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
Immediate Occupancy (N-B)	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not recommended
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	Not recommended	2-D	3-D	4-D	5-D	6-D
Not Considered (N-E)	Not recommended	Not recommended	Not recommended	4-E	Collapse Prevention 5-E	Not rehabilitation



Expected Post-Earthquake Damage State

Operational (1-A)
Backup utility services maintain functions; very little damage. (S-1 & N-A)

Immediate Occupancy (1-B)
The building remains safe to occupy; any repairs are minor. (S-1 & N-B)

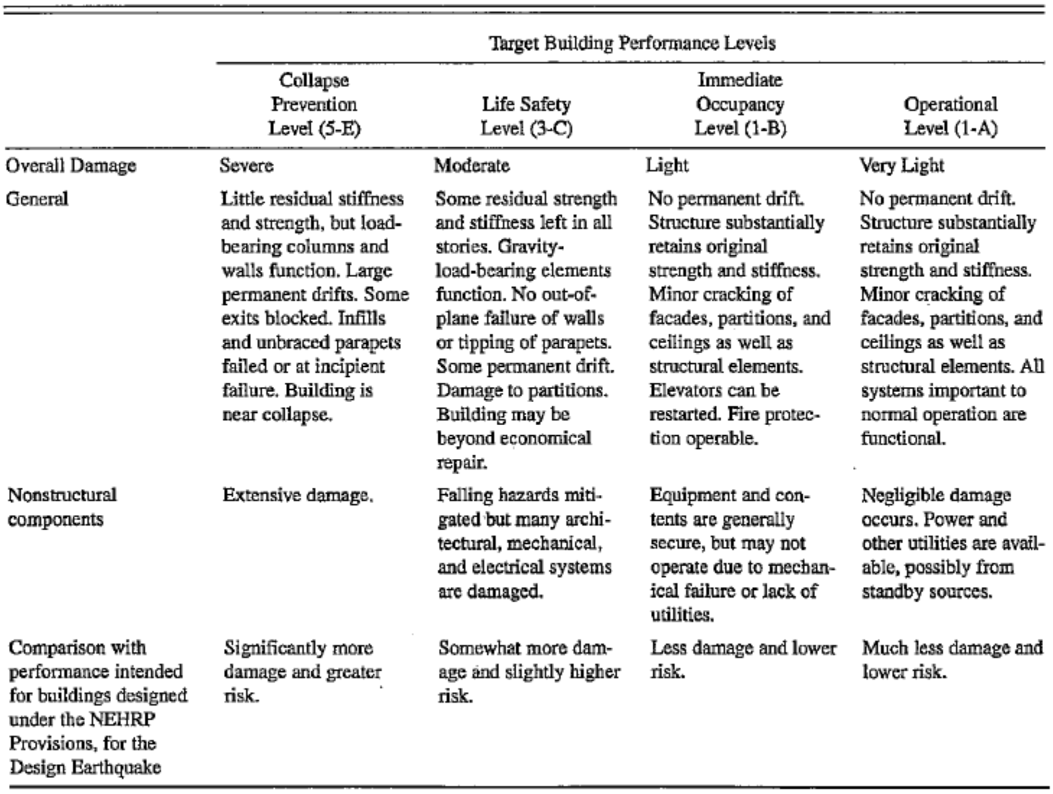
Life Safety (3-C)
Structure remains stable and has significant reserve capacity; hazardous nonstructural damage is controlled. (S-3 & N-C)

Collapse Prevention (5-E)
The building remains standing, but only barely; any other damage or loss is acceptable. (S-5 & N-E)

Figure 3.1. Description of Target Building Performance Levels.

DRAFT

Table 3.2. Damage description for target Building Performance Levels.



	Target Building Performance Levels			
	Collapse Prevention Level (5-E)	Life Safety Level (3-C)	Immediate Occupancy Level (1-B)	Operational Level (1-A)
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed under the NEHRP Provisions, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Less damage and lower risk.	Much less damage and lower risk.

2.2.2 Rehabilitation objectives

A seismic *Rehabilitation Objective* consists of one or more rehabilitation goals. Each goal consists of the verification of a target *Building Performance Level* with an *Earthquake Hazard Level*. These goals are represented as a matrix in the Table 3.3 and are selected considering basic, enhanced, or limited objectives defined as follow:

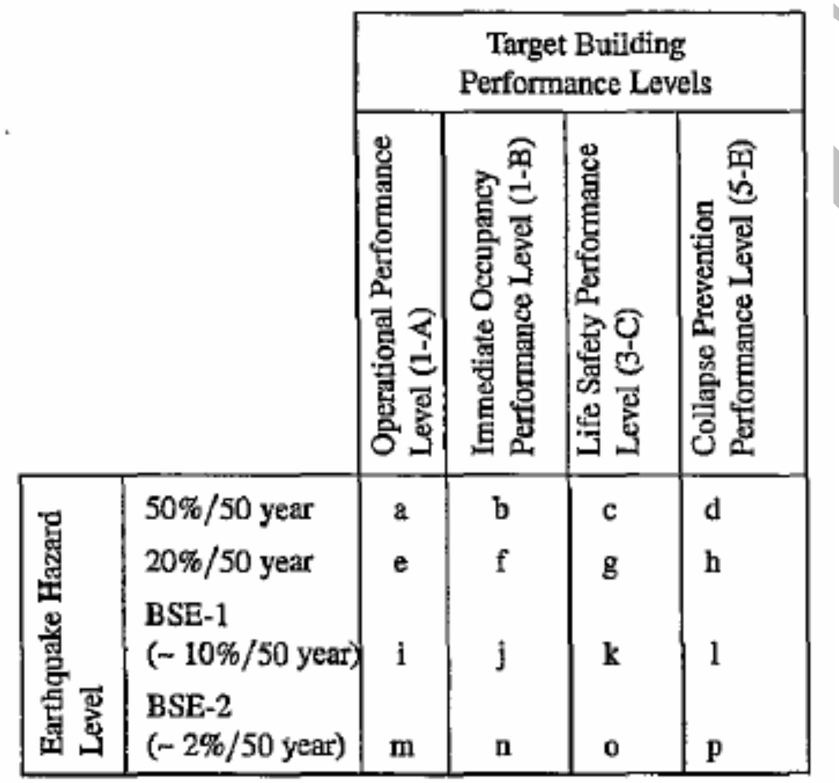
- *Basic Safety Objective* (k and p). The *Basic Safety Objective* (BSO) is a *Rehabilitation Objective* that achieves the dual rehabilitation goals of *Life Safety Building Performance Level* (3-C) for the BSE-1 *Earthquake Hazard Level* (k) and of *Collapse Prevention Building Performance Level* (5-E) for the BSE-2 *Earthquake Hazard Level* (p). Buildings meeting the BSO are expected to experience little damage from relatively frequent, moderate earthquakes, but significantly more damage and potential economic loss from the most severe and infrequent earthquakes.

- *Enhanced Rehabilitation Objectives* (m, n or o; p and i or j; k and p and a, b, e or f). Rehabilitation that provides better building performance than BSO is termed an *Enhanced Objective*. It shall be achieved using one or both of the following two methods: a) by designing for target *Building Performance Levels* that exceed those of the BSO at the BSE-I *Earthquake Hazard Level*, the BSE-2 hazard level, or both (designing for higher target *Building Performance Levels*); b) by designing for the target *Building Performance Levels* of the BSO using an *Earthquake Hazard Level* that exceeds either the BSE-I or BSE-2 hazard levels, or both (designing using higher Earthquake Hazard Levels).

DRAFT

- *Limited Rehabilitation Objectives* (k alone; p alone; c, d, g, h or l alone). Rehabilitation that provides building performance lower than that of the BSO is termed *a Limited Objective*. It shall be achieved using *Reduced Rehabilitation* (that addresses the entire building but uses a lower seismic hazard or lower target *Building Performance Level* than the BSO) or *Partial Rehabilitation* (that addresses a portion of the building without rehabilitating the complete lateral force-resisting system).

Table 3.3. Rehabilitation Objectives



		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake Hazard Level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~ 10%/50 year)	i	j	k	l
	BSE-2 (~ 2%/50 year)	m	n	o	p

2.2.3 Acceptance criteria for evaluation of performance levels

Prior to selecting component acceptance criteria, each structural component shall be classified as primary or secondary and each action shall be classified as deformation-controlled (ductile) or force-controlled (non-ductile). All primary and secondary components shall be capable of resisting force and deformation actions within the applicable acceptance criteria of the selected performance level. The generalized force versus deformation curves (Figure 3.2) is used to specify component modelling and acceptance criteria for deformation-controlled actions in any of the four basic material types:

- linear response is depicted between initial point A and an effective yield point B;
- the slope from point B to point C is typically a small percentage (0%-10%) of the elastic slope, and is included to represent phenomena such as strain hardening;
- point C has an ordinate that represents the strength of the component, and an abscissa value equal to the deformation at which significant strength degradation begins (line CD);

DRAFT

- beyond point D, the component responds with substantially reduced strength to point E; at deformations greater than point E, the component strength is essentially zero.

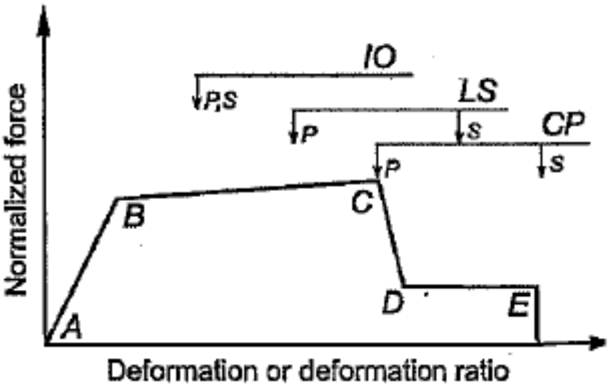


Figure 3.2. Generalized Component Force-Deformation Relations for Depicting Modelling and Acceptance Criteria.

For the definition of the force-deformation curve, both linear and nonlinear procedures can be used. If linear procedures are used, capacities for deformation-controlled actions shall be defined as the product of m-factors and expected strengths, while capacities for force-controlled actions shall be defined as lower-bound strengths. If nonlinear procedures are used, component capacities for deformation-controlled actions shall be taken as permissible inelastic deformation limits, and component capacities for force-controlled actions shall be taken as lower-bound strengths.

2.3 Eurocode 8 (Design of structures for earthquake resistance)

2.3.1 Part 1: General rules, seismic actions and rules for buildings

In the EC8, structures in seismic regions shall be designed and constructed in such a way that the following performance requirements are met, each with an adequate degree of reliability:

- *No-collapse requirement.* The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events (danger for the safety of people is very limited). The design seismic action is expressed in terms of: 1) P_{NCR} (probability of exceedance in 50 years of the reference seismic action for the no-collapse requirement) or T_{NCR} (correspondent return period); 2) the importance factor γ_I, which relates to the consequences of a structural failure, and multiplies the reference seismic action. The values of P_{NCR} or T_{NCR} for use in a country may be found in its National Annex. The recommended value is P_{NCR}=10%, to which it corresponds T_{NCR}=475 years.
- *Damage limitation requirement.* The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence than the design seismic action (no-collapse), without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken

DRAFT

into account for the damage limitation requirement has a probability of exceedance P_{DLR} in 10 years (return period T_{DLR}). In the absence of precise hazard information, a reduction factor may be applied on the design seismic action to obtain the seismic action for the verification of the damage limitation requirement. The values of P_{DLR} or T_{DLR} for use in a country may be found in its National Annex. The recommended value is P_{DLR}=10%, to which it corresponds T_{DLR}=95 years.

2.3.2 *Part 3: Assessment and retrofitting of buildings*

The scope of EC8-Part 3, is:

- to provide criteria for the evaluation of the seismic performance of existing building structures;
- to describe the approach in selecting necessary corrective measures;
- to set forth criteria for the design of retrofitting measures (i.e. conception, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural elements).

This Standard covers the seismic assessment and retrofitting of buildings made of the more commonly used structural materials: concrete, steel and masonry. Although the provisions of this Standard are applicable to all categories of buildings, the seismic assessment and retrofitting of monuments and historical buildings often requires different types of provisions and approaches, depending on the nature of the monuments.

The fundamental requirements refer to the state of damage in the structure, defined through three *Limit States* (LS), namely:

- *Near Collapse* (NC). The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity.
- *Significant Damage* (SD). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.
- *Damage Limitation* (DL). The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures.

The National Authorities decide whether all three *Limit States* shall be checked, or two of them, or just one of them. The appropriate levels of protection are achieved by selecting, for each of the Limit States, a return period for the seismic action. The return periods ascribed to the various *Limit States* to be checked in a country may be found in its National Annex. The protection normally considered appropriate for ordinary buildings refers to the following values for the return periods:

DRAFT

- Near Collapse (NC): 2.475 years (occurrence probability of 2% in 50 years).
- Significant Damage (SD): 475 years (occurrence probability of 10% in 50 years).
- Damage Limitation (DL): 225 years (occurrence probability of 20% in 50 years).

Compliance with the requirements may be achieved by different methods of analysis and verification procedures, as appropriate for the different structural materials; in particular, for masonry buildings, non-linear incremental static analysis is the most reliable in many cases.

For the verification of the structural elements, a distinction is made between 'ductile' and 'brittle' ones. In non-linear procedures, the former shall be verified by checking that demands do not exceed the corresponding capacities in terms of deformations, while the latter shall be verified by checking that demands do not exceed the corresponding capacities in terms of strengths.

For the calculation of the capacities of ductile or brittle elements, mean value properties of the existing materials shall be used as directly obtained from in-situ tests and from the additional sources of information, appropriately divided by the confidence factors, accounting for the level of knowledge attained. Nominal properties shall be used for new or added materials.

Some of the existing structural elements may be designated as "secondary seismic" and shall be verified with the same compliance criteria as primary seismic ones, but using less conservative estimates of their capacity than for the elements considered as "primary seismic".

Regarding the definition of performance limit states on the capacity curve, they are defined by conventional limits directly on the overall capacity curve, thus without a verification of acceptance criteria in each structural element. In particular, for existing masonry buildings they are defined on the basis of the following criteria:

- *Damage Limitation* (DL): the capacity for global assessment is defined as the yield point (yield force and yield displacement) of the idealized elasto-perfectly plastic force-displacement relationship of the equivalent Single-Degree-of-Freedom system.
- *Near Collapse* (NC): global capacity at the Near Collapse limit state may be taken equal to the ultimate displacement capacity taken as the roof displacement at which total lateral resistance (base shear) has dropped below 80% of the peak resistance of the structure, due to progressive damage and failure of lateral load resisting elements.
- *Significant Damage* (SD): global capacity at the Significant Damage limit state may be taken equal to ¾ of the ultimate displacement capacity (NC).

2.4 Italian norms

As already stated in §2, a number of Norms and Guidelines have been produced in Italy on existing and monumental buildings. It is worth noting that, following the Eurocode approach, since 2008 the structural assessment of existing buildings is fully included in the general code for constructions. In particular, the following approach is followed:

DRAFT

- the general methods and approaches for the structural assessment of existing buildings are described in Chapter 8 of the Italian "Technical Rules for Constructions", issued in 2008 and then revised in 2018 (shortly named NTC 2018);
- specific instructions for the application of the "Technical Rules for Constructions" are given in an explanatory Circular of the Ministry of the Public Works ("Instructions for the application of the «Updated Technical Rules for Constructions»", shortly named Circolare 2018);
- within the framework defined in Chapter 8 of NTC2008, specific instructions for monumental buildings are reported in the "Guidelines for seismic risk assessment and reduction of the cultural heritage, with reference to the Technical Rules for Constructions", 2011 (herein named Italian Guidelines).

The whole of these standards and guidelines is a legacy of knowledge deriving from the Italian seismic recent experiences (just like the Umbria earthquake of 1997, the Molise earthquake of 2002 and the Abruzzo earthquake of 2009).

2.4.1 *General framework*

Being an organic document, the "Technical Rules for Constructions" provide a general framework that is valid for both the design of new buildings and the assessment of existing ones. In this framework, buildings are divided in categories depending of their social and strategic relevance. With particular reference to seismic actions, the idea is to provide a graduated protection depending on the role of the building within the society. The following classes are considered:

- Class I: buildings where the people might be present only occasionally; farm buildings;
- Class II: regular buildings without crowds, without environmental hazardous contents and activities, and without public and strategic functions (e.g. residential buildings);
- Class III: crowded buildings and buildings with environmental hazardous contents and activities (e.g. schools, public offices, theatres, chemical industries…);
- Class IV: buildings with relevant public and strategic functions, with reference to civil protection activities also.

In this framework, Chapter 8 provides detailed instructions for existing buildings, that are here summarized:

- the structural assessment of an existing building should be based on a quantitative procedure, aiming at defining the entity of the actions that the structure is able to support with the minimum safety level required;
- structural assessment should be performed when at least one of the following conditions is fulfilled:
    - evident reduction of the strength/deformation capacity of the structure related to material degradation
    - significant deformations (also related to foundation problems)

DRAFT

  - damage induced by either environmental (earthquake, wind, snow and temperature) or accidental (impacts, fire, explosion) actions
  - evidence of design or building mistakes
  - changes in use with significant load increasing
  - carrying out of non-structural interventions (in the event they interact with structural elements modifying their strength of stiffness)
  - strengthening interventions
- the safety level of an existing building is quantified as the ratio ζ between the highest action that can be borne by the structure and the highest action considered for the design of a new building;
- if it is assessed that the building is “unsafe” in its current status, there are two solutions: a) change of use; b) structural interventions;
- three types of structural interventions are considered:
  - local reparations: interventions aiming at repairing individual structural elements, not changing the overall structural behaviour of the structure;
  - improvement interventions: interventions aiming at increasing the structural safety of the building, without necessarily achieving the safety levels considered by the code;
  - retrofitting interventions: interventions aiming at increasing the structural safety of the structure, achieving the safety level considered by the code;
- in case of improvement and retrofitting interventions, the structural assessment should be performed before and after the intervention: in the case of improvement interventions, the aim is to quantify the increasing in the safety level obtained with the intervention; in the case of retrofitting interventions, the aim is to demonstrate the achievement of the required safety level;
- retrofitting interventions are mandatory only when one of the following conditions is fulfilled:
  a) building superelevation;
  b) building enlargement (when the new part is connected to the existing one and change its structural behaviour);
  c) structural modifications (when they significantly change the structural behaviour of the building);
  d) changes in use (when they produce an increasing of the static loads larger than 10% or when they involve strategic functions);
- with reference to the seismic actions, in the case of retrofitting interventions the following safety levels should be achieved:
  - in cases a), b) and c): $\zeta E = 1$
  - in case d): $\zeta E = 0.8$

DRAFT

- with reference to the seismic actions, in the case of improvement interventions the following safety levels should be achieved:
    - for "class III" buildings used as school and for "class IV" buildings: $\zeta_E = 0.6$
    - in any case, improvement interventions should provide an increasing of $\Delta\zeta E>0.1$;
- for heritage buildings, retrofitting interventions are never mandatory.

The last point is very important. In agreement with comma 4/art. 29 of the Italian "Code for heritage and landscape assets" (DLgs 22 gennaio 2004, n. 42), it is stated that for heritage buildings it is possible carry out improvement interventions only, provided that the achieved safety level is always quantified. This are the main innovations of the Italian code for heritage buildings:

- it is always mandatory the quantitatively assess the safety of existing buildings;
- for heritage buildings, it is not mandatory the achieve the same safety level of new buildings;
- for heritage buildings, the design of improvement interventions is encouraged, but their effectiveness should be always demonstrated by a comparison of the safety level before and after the intervention.

Given that safety assessment of existing buildings is always mandatory, Italian code introduces another relevant innovation in structural assessment procedures. In particular, it states that the reliability of the structural models used to evaluate the safety of the structure depends on the level of knowledge of the construction. For this reason, the lower is the knowledge of the construction, the simpler are the models employed and the lower is the calculated capacity. This is obtained through the introduction of "confidence factors", quantifying our knowledge and reducing the mechanical parameters of the materials.

2.4.2 *Instructions for monumental buildings*

Within the framework of the "Technical Rules for Constructions" (NTC 2018), the Italian Guidelines (2011) define the methods to perform the structural assessment of heritage buildings and discuss the design of structural improvement interventions.

As far as assessment methods are concerned, the Italian Guidelines (2011) introduce a new approach for the evaluation of the seismic safety through a multilevel path which tries to coordinate the phases of investigation and assessment with different grades of resolution depending on different possible objectives. In particular, three levels of investigation and assessment are defined:

- LV1: territorial-scale seismic evaluation through simplified approaches;
- LV2: seismic evaluation to be used in case of local interventions on a building;
- LV3: accurate evaluation of the seismic safety of a building.

DRAFT

The first level assessment, LV1, is oriented to highlight, on a regional scale, critical situations in terms of seismic vulnerability, to provide a risk classification and to define priority lists for further investigations aimed at the conservation of architectonical heritage. Adopting a force-based approach, this level relies on different simplified models related to the different building types (churches, palaces, towers, arches, etc…). In general, such methods require integrating a limited number of geometrical and mechanical parameters with qualitative data derived from visual tests, construction features and stratigraphic surveys.

The second level assessment, LV2 is aimed at evaluating the seismic safety of the building when local interventions on single parts are carried out (it is important to underline that the LV2 can be used only when local interventions do not modify the structural behaviour of the building). It is usually based on the study of out-of-plane mechanisms and relies on the use of limit equilibrium analysis.

Lastly, the LV3 is based on the global structural response of the building in order to define the values of acceleration leading the structure to each limit-state. In this case, the displacement-based approach is adopted, for which the global behaviour is governed by the in-plane capacity of the walls discretized in panels where the nonlinear response is concentrated.

In the Italian Guidelines (2011) great attention is paid to the knowledge of the building and to its influence of the structural models and analyses. It is highlighted that the knowledge of a heritage buildings should consider many factors and many tools, ranging from the analysis of historical documents, to the digital survey and the experimental tests on the materials.

As far as interventions are concerned, the Italian Guidelines (2011) provide a set of principles and ideas that are here summarized: -

- there is usually more than one possible intervention for each kind of pathology or form of vulnerability, with diverse characteristics in terms of effectiveness, invasiveness, reversibility, compatibility, durability and cost;
- the choice of solutions is the primary concern of the project and must be chosen after careful examination of the specific situation and the effectiveness of the proposed solution must be verified;
- the possibility of falling back on provisional methods should not be overlooked, which due to their striking intrinsic reversibility, appear interesting with respect to preservation and after careful evaluation, may also result to be the definitive solution for historic buildings;
- the interventions are classified depending on their functions: interventions to improve connections; interventions to reduce thrust of vaulted arches and their strengthening; interventions to reduce excessive flexibility of floors and their consolidation; interventions for roof coverings; interventions for increasing the strength of masonry elements; interventions for pillars and columns; interventions on non-structural elements; interventions on foundations.

The Italian Guidelines (2011) propose many different types of traditional and innovative intervention techniques, with a great attention paid to reversibility, structural and chemical compatibility and to conservation issues. In general, a deep criticism of most of reinforced concrete interventions carried out in the past and still today can be noted.

DRAFT

DRAFT

2.5 Moldovan norms

2.5.1 General framework

The structural design rules in force on the territory of the Republic of Moldova are presented in the Construction Documents Catalogue:

http://www.ednc.gov.md/normative_in_constructii/catalogul_normativelor

The maintenance of the respective document is ensured annually by the Ministry of Economy and Infrastructure.

Presently, the system of building normative documents (SDNC - Sistemul De Documente Normative În Construcţii) of the Republic of Moldova consists of 2615 normative documents (Figure 3.3). Most of them are norms and standards of former U.R.S.S. and R.S.S.M. Their application was allowed on the territory of the Republic of Moldova by letter of the Ministry of Architecture and Construction of the Republic of Moldova no. 03-05/340 "Regarding the functioning of norms under construction on the territory of the Republic of Moldova" (1993). This letter allowed the application of the normative documents of former U.R.S.S. and R.S.S.M., until they would have been cancelled or specified.

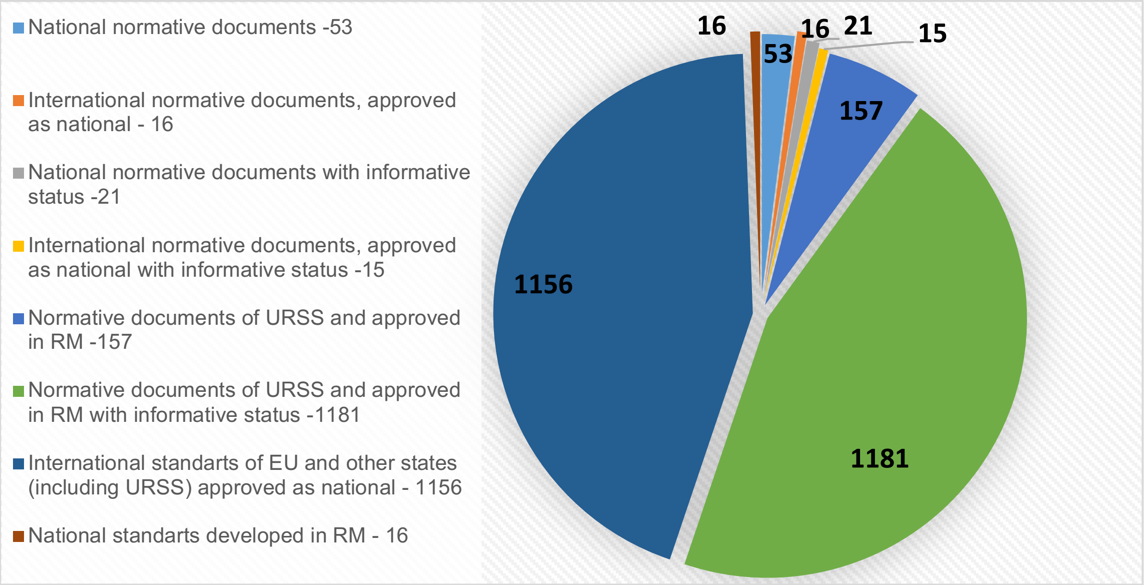


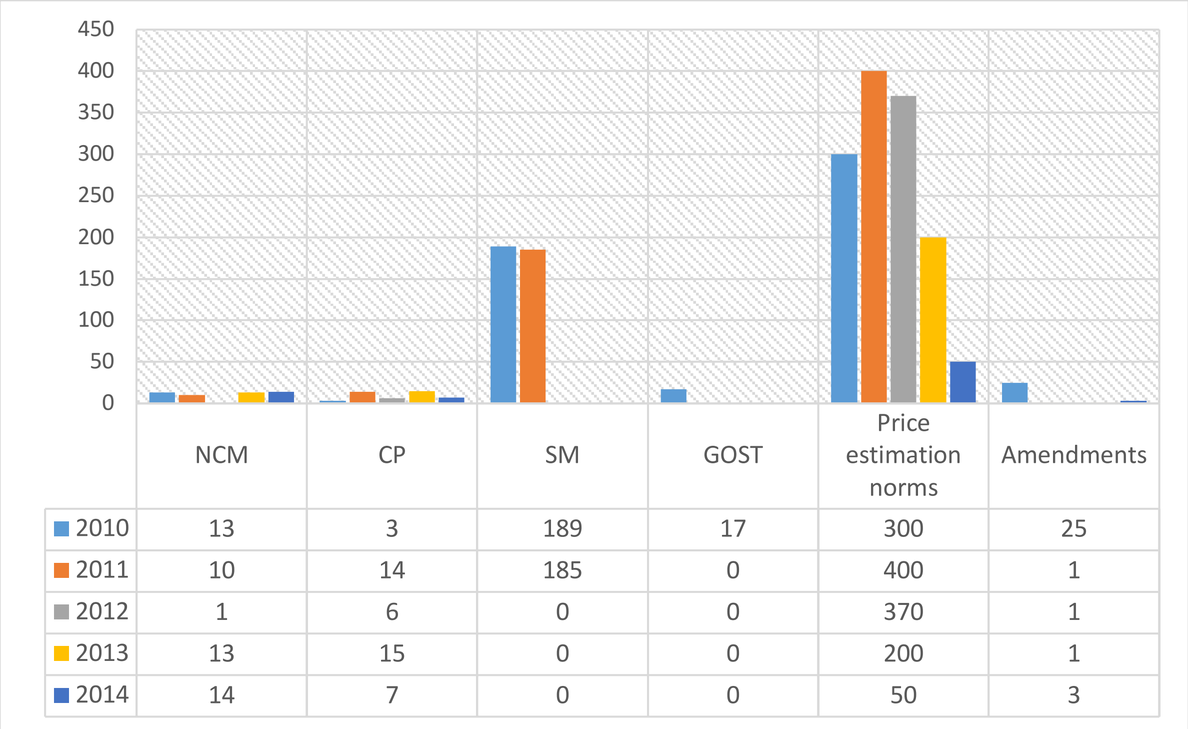
Figure 3.3. Structure of normative documents in Republic of Moldova.

Since the proclamation of the independence of the Republic of Moldova in 1990, SDNC development evolved differently. It was mainly based on the elaboration of national norms, by adopting the normative documents of the Russian Federation, Belarus, Ukraine (Figure 3.4 reflects the dynamics of developing normative documents in constructions in 2010-2014). This development was determined by the following factors:

- the similarity of normative systems in constructions, determined by a common historical normative basis (normative documents of the former USSR);

- the accessibility of the normative documents of the Russian Federation, Belarus, Ukraine (own to the existence of cooperation relations and agreements with these countries and the relative simplicity of use and understanding of the norms of these countries, because they exist in Russian).
- The reduced access to application of European standards in the field of construction.

In general, this complex and fragmented normative framework makes the analysis of the principles and the ideas of Moldovan norms in the field of constructions very difficult.



	NCM	CP	SM	GOST	Price estimation norms	Amendments
2010	13	3	189	17	300	25
2011	10	14	185	0	400	1
2012	1	6	0	0	370	1
2013	13	15	0	0	200	1
2014	14	7	0	0	50	3

Figure 3.4. Dynamics of development of Moldovan normative documents in construction 2010-2014.

2.5.2 *Moldovan laws and norms in context of existing buildings*

The assessment of existing buildings in Republic of Moldova is regulated by Government Decision Nr. 936 from 16.08.2006 “Regulation on technical expertise in construction” (2006), whose aim is:

- to assess the technical condition of the constructions and bring them in accordance to the standards;
- to design restoration works in closed, preserved, and abandoned buildings;
- to design building reconstruction and/or extension;
- to design consolidation of deformed structures;
- to manage the changes of use.

In addition, it is necessary to mention another specific code NCM A09.02-2005 “Technical maintenance, repair and reconstruction of housing, common and socio-cultural buildings”. This code establishes the structure and the mode of functioning of technical maintenance of buildings. The code is mandatory for all

DRAFT

DRAFT

economic entities and persons who performs maintenance work. Furthermore, the annexes of this code regulate other aspects like:

- Annex 1 - Minimum term for efficient exploitation of building
- Annex 2 - Minimum term for efficient exploitation of building elements (foundation, walls, etc.)
- Annex 3 – List of main maintenance works
- Annex 4 – Inspecting periodicity of building elements
- Annex 5 – Repair term of defected elements
- Annex 6 – List of current reparation work
- Annex 7 – List of works in apartments that are made by the owners.
- Annex 8 - Work financed from the assets allocated for building capital repair
- Annex 9 - The building technical inspection register

Other rules on existing buildings are provided in the national normative code regulating the design of new structures NCM F02.02.-2006 (2006). In the first chapter, in fact, clause 1.3 states that "*The previsions of this code are mandatory for all economic entities for designing new buildings, assessment, rehabilitation and reconstructions of existing buildings*". Concerning concrete structures, does not even mention existing structures. Concerning steel structures, in Chapter 20 "Additional requirements for the design of structures during reconstruction" of the national code SNiP II-23-81 (1981), additional rules for reconstruction works are provided. The code stresses the following topics for existing steel buildings: yield strength of material; material safety coefficient for structures build in different periods of time; methods of safety assessment; cases where structure should be consolidated and cases where consolidation can be omitted. NCM F.03.02-2005 (2005) regulates the design of masonry structures. In chapter 6.5 "Reconstruction of buildings with masonry walls" reference is made to existing structures, but the code does not provide a clear assessment method. Chapter 6.5 specifies:

- what should contain a technical expert report
- scope of reconstruction
- determination of seismic resistance of building
- the "most efficient" consolidation methods of masonry walls

It can be observed that, in most of these documents, reference is made to "reconstruction" rather than "strengthening".

2.5.3 *Moldovan norms and codes in context of heritage buildings*

Basis of the national protection system of heritage buildings were adopted in 1993 by the parliament of the Republic of Moldova. Law nr. 1530 from 22.06.1993 "Regarding Protection of Monuments," states basic principles of protection, conservation and restauration of monuments. Joining UNESCO in 1992, Moldovia made an effort to created good conditions for assimilating the experience of European and International system of heritage protection and adopting or implementing the normative and legislative bases in Republic of Moldova (Plesca & Fagurel, 2017).

Starting with 2010, Republic of Moldova created a national heritage protection system. The system concept was based on ensuring each branch of cultural heritage with a regulatory and institutional framework. From this period on:

- the *Monuments Inspection Agency of Ministry of Culture* began to be operational;
- the *National Council of Historical Monuments* was established;
- any intervention on heritage buildings or sites should be mandatorily approved by the Ministry of Culture (now Ministry of Education, Culture and Research), based on the opinion of the National Council for Historic Monuments, as per amendments to the Law n. 1530/1993;
- for the first time, the Cultural Development Strategy "Cultura – 2020" sets the problems of the national cultural heritage.

As regards to the normative base, from previous chapters it can be noted that existing norms in Moldova do not cover a large range of heritage buildings and opens some questions regarding the normative base assessment and monitoring of heritage structures. All this considered the normative base of structural assessment and monitoring of heritage buildings is a vulnerable point for Republic of Moldova.

DRAFT

3. Methods of modelling and analysis of historic masonry structures

3.1 Modelling issues related to heritage buildings

Linear and non-linear modelling of masonry historic structures represents today one of the most important field of research in civil and conservation engineering area. In spite of the efforts of scientific community, there are still strong uncertainties when dealing with the structural behaviour of masonry, much larger than for steel or reinforced concrete structures. The difficulty in modelling such structures depends mainly on three fundamental problems: the composite nature of masonry, made up of a complex system of blocks and joints inducing a strongly anisotropic response; the mechanical non-linearity of the material, also for low stress values; the geometric complexity of masonry constructions, commonly requiring the adoption of complex 2D and 3D modelling approaches.

With reference to the composite nature of masonry, it is worth noting that the behaviour of masonry strongly depends on both mechanical characteristics of constituents (stiffness and strength of blocks and joints) and rules of construction of the assemblage (as an example, consider a regular masonry made of bricks or an irregular masonry made of stone of variable shape and dimension). The micromechanics of the composite plays a fundamental role in the behaviour of the material, inducing a strong anisotropy in both linear and non-linear response. However, its detailed description usually requires intensive computational efforts and should be thus calibrated to the aims of the analysis.

With *reference* to the mechanical non-linearity of the material, models should balance accuracy and efficiency. This means that they should be able to describe with “sufficient” accuracy the actual behavior of the material with a “reasonable” computational effort. This need is even more significant in the case of performance-based approach, which requires modelling tools capable to evaluate the evolution of the response of entire masonry buildings in the non-linear range up to collapse. Regarding this, despite the efforts of many researchers and the results achieved starting from early Eighties, the tools of analysis today available are not shared by the scientific community and are not widely diffused in engineering practice. On the one hand, there are “theoretical” approaches, able to describe in a very detailed way the behaviour of the material at micro-scale but unable to solve problems at the scale of the construction (due to the complexity and the computational effort required). On the other hand, there are “engineering” approaches that, by describing the behaviour of the material through phenomenological simplified constitutive laws, allow to perform analyses at the scale of the construction but may result unreliable for certain structural and load configurations. Moreover, some of the approaches require only a few number of constitutive parameters, but provide only a rough representation of the structure; on the contrary, some others are able to give an accurate description of the non-linear behaviour, but require a large number of constitutive parameters, some of which do not have a clear physical meaning and are therefore difficult to calibrate.

With reference to the geometric complexity of masonry constructions, it is worth noting that masonry buildings are always complex continuous structures in which it is difficult to recognize a clear structural scheme. Commonly, the construction is made by set of bidimensional structural elements (walls and floors). In some case, such as towers, the walls are so thick that the bidimensional idealization is not possible and 1D simplified models or 3D complex models are even required. In the case of historic masonry buildings, this

DRAFT

issue is even worsened by the variations of structural configuration that may be occurred during the centuries (partial demolitions, integrations and modifications, strengthening and seismic retrofitting interventions). This complexity poses questions on both global model of the structure (for example related to the definition of proper boundary conditions) and analysis of single portions. The afore mentioned issues stress how the choice of suitable modelling strategies requires an accurate preliminary analysis of the asset. The decision-making process should take into account the choice of spatial scale of the analysis and definition of a proper structural scheme.

With reference to the choice of spatial scale, possible scales are: that of the building, that of macroelements (the use of "macroelement" word refers to portions of an architectonic asset – which usually include a set of structural elements - for which it is possible to recognize recurring seismic behaviour) or that of structural elements. In this context, the urban scale is here neglected. These different scales may imply the adoption of different modelling strategies (for example some modelling strategies should be applicable to either structural elements or macroelements, as a function of the computational effort, but not to whole architectonic assets). The choice of "proper" spatial scale depends on the following main factors: the prevailing seismic response (e.g. in some cases, when the box behavior is completely disregarded and the overall response may be traced back to that of single macroelements, it seems justifiable to directly refer to the macroelement scale); the need to analyze specific details with a greater degree of accuracy (e.g. in some case - in addition to an evaluation on the whole asset - it could be necessary to model in detail some specific portions due to damage concentration); the strengthening interventions planned. Concerning this latter point, the insertion of complex systems of tie-rods, aimed to improve the overall seismic response of the building, may require a full-scale model, while the insertion of a tie-rod in an arch may require a simple model of the single arch only. Furthermore, interventions like the strengthening of masonry by FRP or mortar injections may require detailed models of the material, able to provide new equivalent mechanical parameters to be used in full-scale models.

With reference to the definition of the proper structural scheme, this step comes from the synthesis and the critical analysis of several information provided by the knowledge phase. The most relevant issues to be investigated are: geometry and building morphology, urban context (i.e. if isolated or in aggregated), subsequent integrations and modifications occurred, damage pattern. The geometry analysis is addressed to identify possible plan of symmetry or significant directions of the global response in order to reduce the number of degrees of freedom and to pass from 3D to 2D models. A typical example is represented by common configurations of domes or vaults. Of course, it is not always possible to apply these simplifications, for example due to constructive irregularities. Regarding this, Figure 4.1 shows the dome of Hagia Sophia in Istanbul; despite its apparent regularity, this church suffered many damages from earthquakes and several repairs during the centuries, losing its original circular shape.

DRAFT

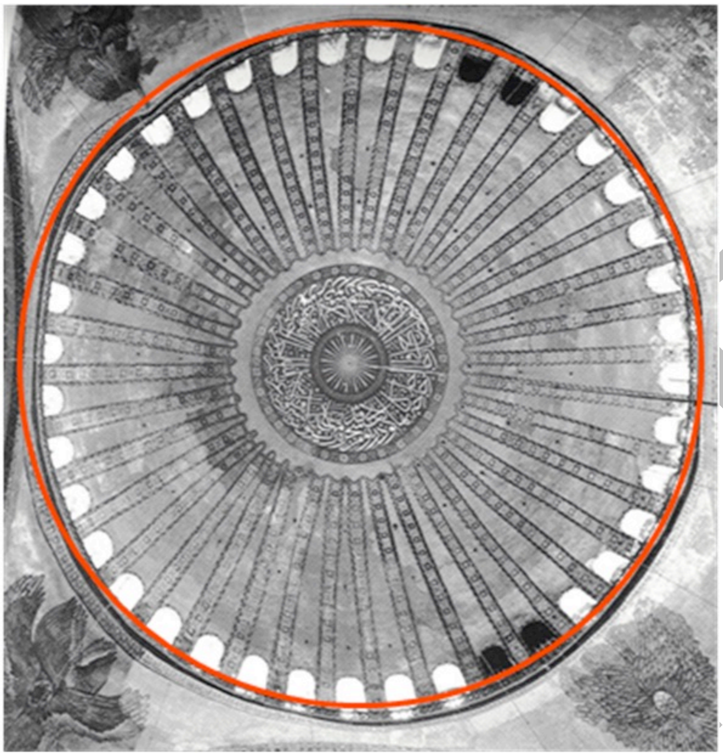


Figure 4.1. Geometrical irregularity of the dome of Hagia Sophia Church in Istanbul.

With reference to the urban context, this aspect is fundamental to define proper boundary conditions (as an example, Figure 4.2 shows the case of a tower enclosed in an aggregate). As well known, the analysis of the subsequent transformations may help to individuate possible discontinuities among different buildings in aggregate.

DRAFT

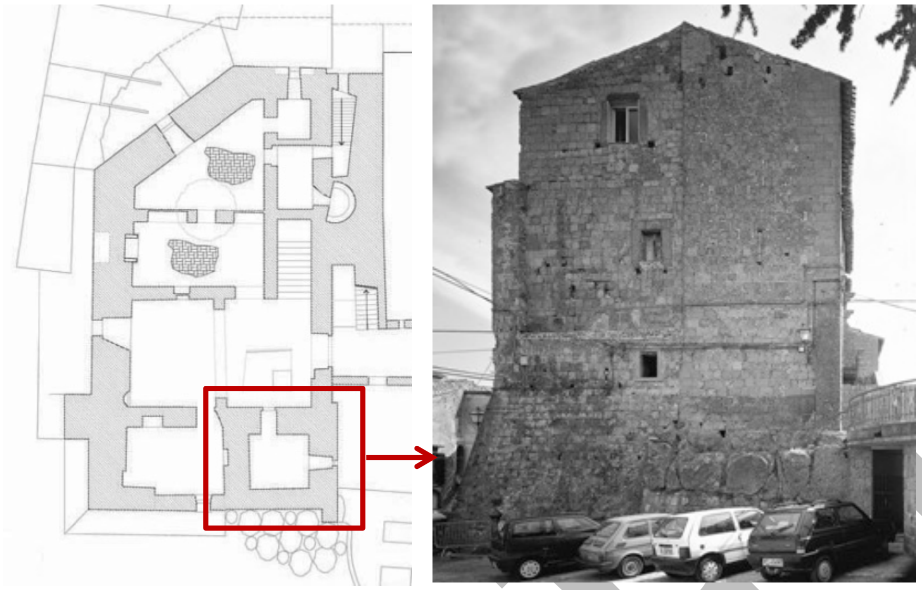


Figure 4.2. Farnesina Tower in Cellere (Italy).

With reference to the damage pattern or the actual configuration of the asset, in some cases it is necessary to compute the effective actual stress state of the structure by taking into account its deformed shape. Figure 4.3 shows a typical example of a vault which suffered a partial collapse and for which the actual configuration is strongly affected by past deformations occurred.

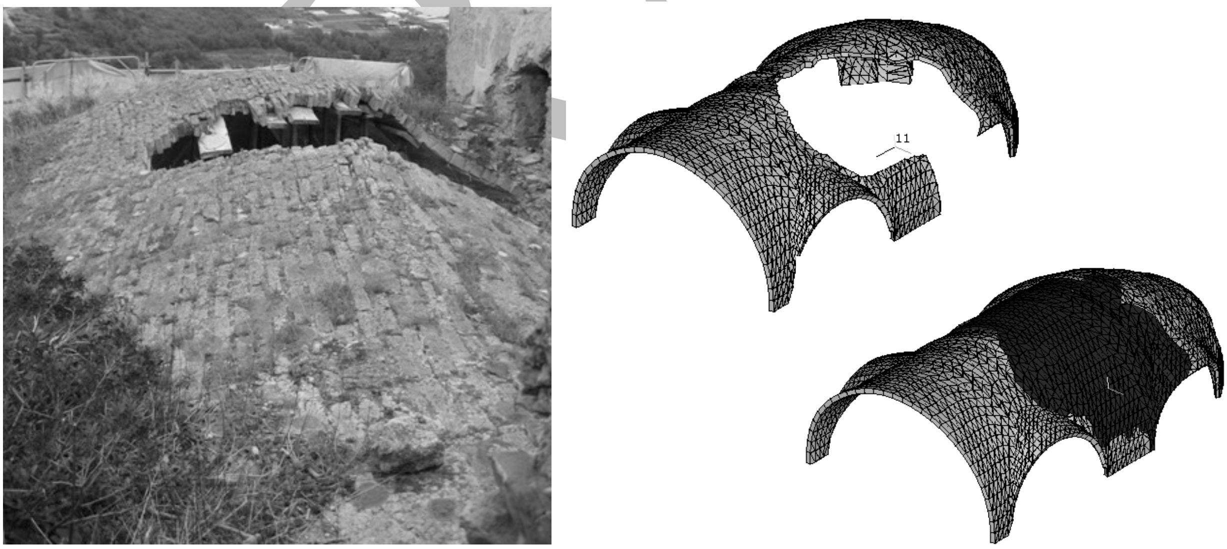


Figure 4.3. Nostra Signora delle Grazie church in Bussana (IM).

In some cases, the analysis of the actual damage state may address the choice of the most suitable and simple modelling strategy. Regarding this point, Figure 4.4 shows the case of a barrel vault characterized by the

DRAFT

same geometry but different damage patterns. Case A and B differ for the hinge positions, but in both cases reference should be made to a 2D model (which in case A is limited to the arch and in case B consider the pier-arch system). In case C, a prevailing flexural behavior of the overall system (due to a beam-like effect), imposes the adoption of a 3D model.

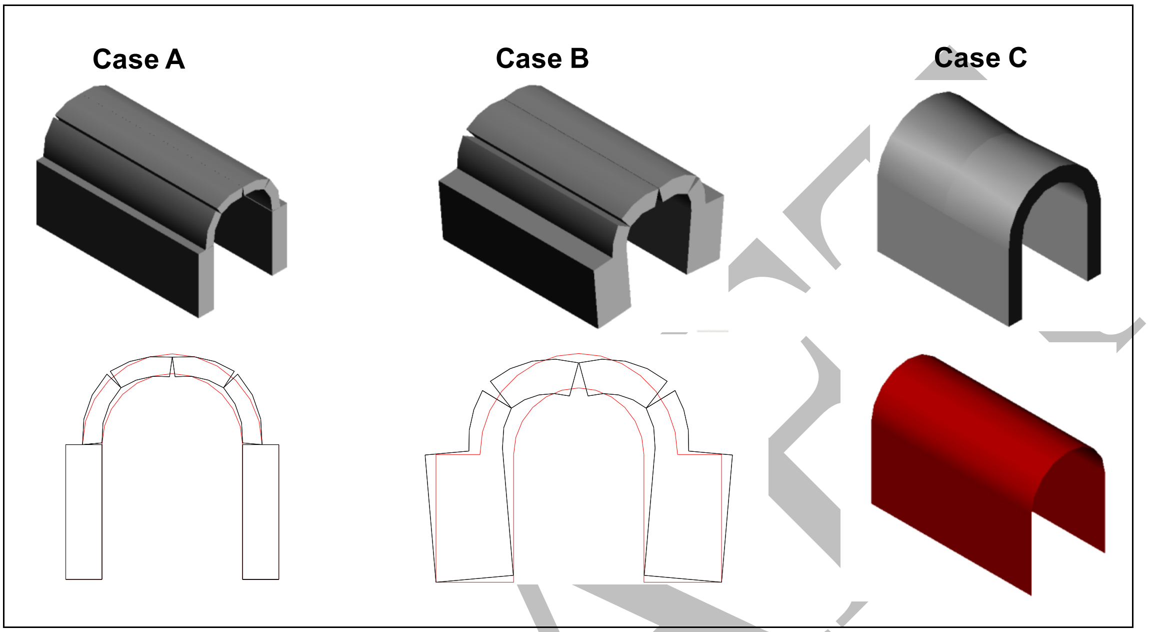


Figure 4.4. Influence of the damage pattern on the choice of structural schemes.

Regarding the choice of the more suitable modelling strategy to be adopted, it is clear that it cannot disregard from a detailed knowledge of the structure and material properties. In fact, lacks in knowledge may lead, on the one hand, to choices which may turn out to be far from the actual seismic response of the structure and, on the other hands, to adopt large safety factors to balance the uncertainties affecting the modelling of the structure. With reference to the first issue, it seems worth noting that, in some cases, the adoption of too simplified assumptions (i.e. due to a poor knowledge of building) may produce severe underestimations of the actual resistance, that are on the safe side but lead to too invasive retrofitting interventions. In general, this means that a maximization of the knowledge of the structure (in term of geometry, constructive details, material properties…) lead to more reliable modelling strategies and to a minimization of interventions. Moreover, it seems important to note that, in some cases, the reliability of the results provided by very detailed models may result strongly affected by the uncertainties on the mechanical parameters which the model is based on. As a consequence, the decision-making process has to be take into account a good compromise between the knowledge obtained and accuracy of the models adopted.

With reference to the modelling issues related to historic buildings, it seems important noting that in some case (in particular in case of very massive masonry structures) effects associated to the soil-structure interaction have to be taken into account in models and seismic analysis.

DRAFT

3.2 Types of damage and classes of buildings

Starting from data provided by earthquake damage survey (based on literature review and direct damage assessment data), the proposed damage classification identifies the most common seismic damage modes which may occur in the cultural heritage assets. It is worth noting that the proposed classification accounts for damage caused by earthquakes only, thus disregards further effects due to environmental deterioration, service conditions, etc. Moreover, although earthquake can cause damage to both structural and non-structural components of buildings, this classification is addressed only to structural ones.

The general criteria adopted for the damage classification are illustrated in the following; a detailed description of each class of damage is then provided in sub-paragraphs from 4.2.1 to 4.2.9.

The proposed classification aims to distinguish the different failure modes that may occur in masonry structures as a function of the direction of the seismic loads (whether in-plane or out-of-plane), the structural element considered (walls, floors or vaults, columns or pillars…) and the type of masonry (small or large block masonry).

The nine damage classes are summarized in Table 4.1.

Table 4.1. Damage classes for architectonic assets.

Damage class		Description
A	Damage to in-plane loaded walls	This class collects damage to piers, spandrel beams and whole walls loaded in their plane. Different types of damage are considered: diagonal cracking, bed joint sliding, rocking, cracks in constructive joints.
B	Damage to out-of-plane loaded walls	In this class, partial and global overturning of walls or masonry elements are collected. Both purely out-of-plane and mixed in-plane/out-of-plane motions of masonry blocks are considered.
C	Damage to masonry elements subjected to combined axial and bending loads	Two types of damage are considered: damage to slender structures (towers, bell-towers, …) and damage to columns and pillars in buildings. These structures may fail in compression due high compressive stresses (produced by gravity loads or high vertical seismic loads) and bending effects.
D	Damage to in-plane loaded arches (or vaults)	In this class, arch structures loaded in their vertical plane are considered. Damage usually involves the arch-and-piers system and occurs by means of tensile cracks which tend to turn the structure into a set of blocks.
E	Local damage of masonry	The damage considered in this class does not interest structural elements, but it is always restricted to single parts of masonry continuum. Different types of damage are included: cracks and spalling in massive masonry structures due to hydrostatic thrust; detachment of external masonry leaf in multi-leaf walls, closure of openings; pounding of masonry due to floor of roof beams.
F	Rocking of single or multiple blocks	In this class, damage to standing out elements is considered. Usually, due to their dimensions and boundary conditions, such

DRAFT

		elements are slightly compressed and tend to behave as single or multiple rocking blocks.
G	Unthreading or disconnection of structural elements of roofs and floors	In this class, damage to connections of timber beams and wooden structures is considered. This type of damage tends to produce failure of roofs and floors.
H	Drift of vaults in their horizontal plane	Damage to vaults subjected to in-plane movements of their abutments are considered. These movements usually produce shear cracking and local instability of vaults.
I	Damage to domes	Typical three-dimensional damage is produced on domes due to their spatial configuration. Shear, out-of-plane and arch behaviours may coexist, producing complex damage states.

Since each basic class may include several damage patterns and cracks morphologies, some damage sub-classes have been introduced as illustrated in Table 4.2 (also in this case, for a detailed description of these sub-classes, reference is made to paragraphs from 4.2.1 to 4.2.9).

Table 4.2. Damage classes and sub-classes.

Damage class		Damage sub-class
A	Damage to in-plane loaded walls	A-a: generic cracking
		A-b: cracks in piers
		A-c: cracks in spandrels
		A-d: cracks in structural gaps
B	Damage to out-of-plane loaded walls	B-a: single block overturning
		B-b: multi-blocks overturning
C	Damage to masonry elements subjected to combined axial and bending loads	
D	Damage to in-plane loaded arches (or vaults)	
E	Local damage of masonry	E-a: partial collapse of external leaf of masonry walls
		E-b: damage to infill walls, cavities or structural gaps
		E-c: local damage to masonry in correspondence of structural elements of floors/roofs or bolts of tie-rods.
F	Rocking of single or multiple blocks	
G	Unthreading or disconnection of structural elements of roofs and floors	G-a: damage in correspondence of supports of wooden floors and roofs
		G-b: damage in correspondence of r.c. rods or floors/roofs
		G-c: damage to structural elements
H	Drift of vaults in their horizontal plane	

DRAFT

I	Damage to domes

3.2.1 *Class A - Damage to in-plane loaded walls*

This class mainly collect the in-plane damage of walls, which progressively lead to the loss of their bearing capacity.

The earthquake damage observation shows that, if walls are characterized by regular geometry and standard size openings, cracks and failures in in-plane loaded masonry walls are frequently concentrated in clearly recognizable portions of the structure, named as piers and spandrels. Piers, which are the principal vertical resistant elements for both dead and seismic loads, are those parts of walls between two horizontally aligned openings, whereas spandrels, which are secondary structural elements coupling piers in case of seismic loads, are intended to be those parts of walls between two vertically-aligned openings. Some simplified sketches with damage concentration in piers or spandrels elements are depicted in Figure 4.5.

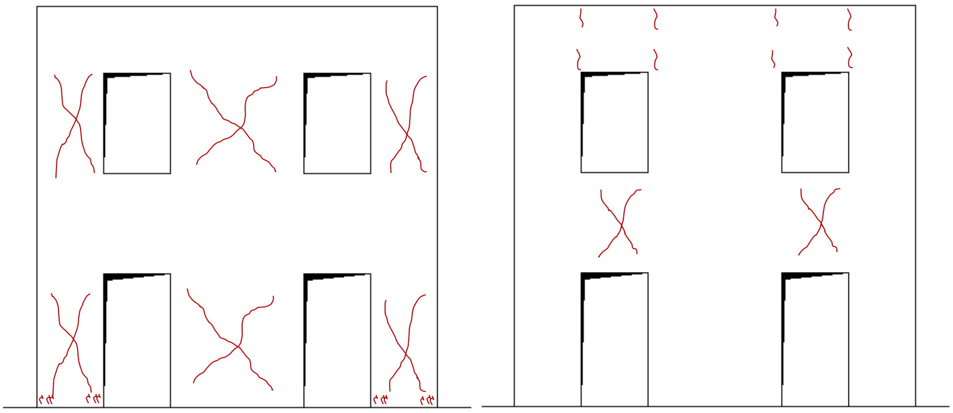


Figure 4.5. Failure mode of a masonry wall with a damage concentration in pier (a) or spandrel elements (b) respectively.

Moreover, generic cracks in masonry walls may be surveyed without a prevailing concentration on piers and spandrels elements whenever openings are not regularly arranged (Figure 4.6). Indeed, the systematic identification of piers and spandrels as a function of the architectonic assets may appear to be quite ambiguous.

DRAFT

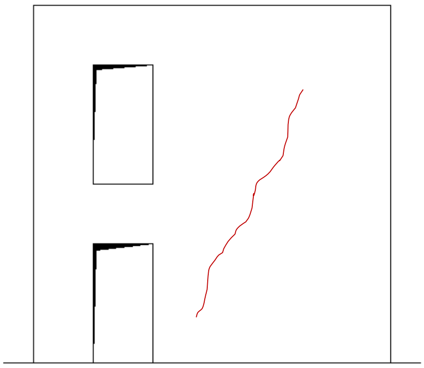


Figure 4.6. Failure mode of a masonry wall with generic pseudo-diagonal cracks.

Historical buildings are often the result of several successive building phases, which might cause the presence of structural gaps (e.g. closure of pre-existing openings), depending on the quality of connections among these different portions of the building. As a consequence, the latter gaps constitutive preferential path for cracks.

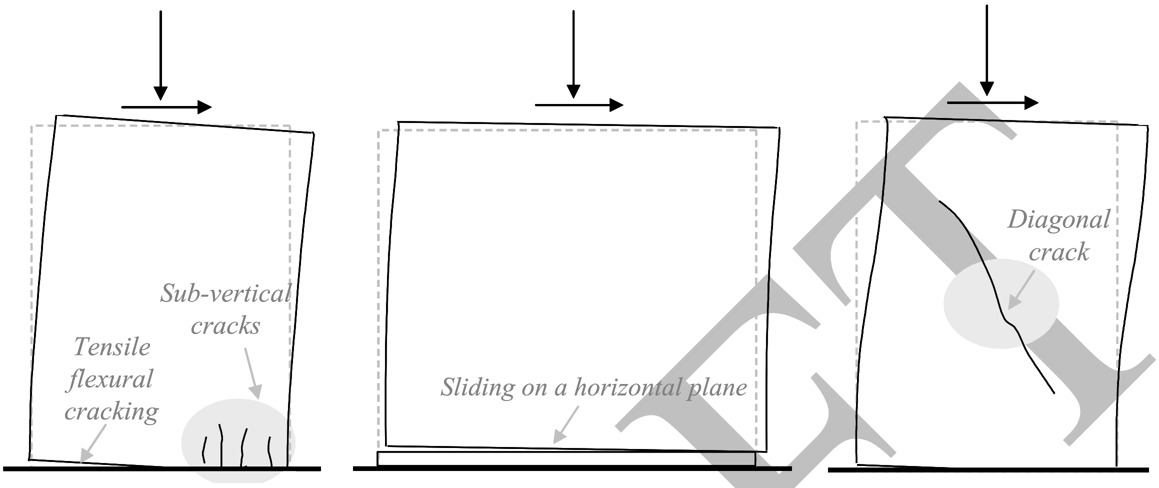
According to the main cases above mentioned and in order to collect the most recurring seismic responses testified by the earthquake damage observation, the following main sub-classes have been introduced in class A:

- A-a: cracks in piers;
- A-b: cracks in spandrels;
- A-c: generic cracking;
- A-d: cracks in structural gaps.

It is important to stress how the sub-classes that have been introduced aim to classify recurring global failure modes (associated to the in-plane behaviour) at scale of masonry walls. Actually, damage in single structural elements (i.e. piers and spandrels) may occur according to different local crack pattern or morphologies. Regarding this issue, note that observations of seismic damage to complex masonry walls, as well as laboratory experimental tests, have shown that a masonry panel subjected to in-plane loading may present two typical types of behaviours, i.e. flexure and shear, associated to different failure modes (Figure 4.7): Rocking and Crushing (flexural behaviour); Sliding Shear Failure and Diagonal Cracking (shear behaviour). In particular, in the case of flexural behaviour, this may involve two different modes of failure. If the applied vertical load is low with respect to compressive strength, the horizontal load produces tensile flexural cracking at the corners (Figure 4.7-a), and the pier starts to behave as a nearly rigid body rotating about the toe (Rocking). If no significant flexural cracking occurs, owing to a high applied vertical load, the pier is progressively characterized by a widespread damage pattern, with sub-vertical cracks oriented towards the more compressed corners (Crushing). In both cases, the collapse is obtained by failure at the compressed corners. In case of shear behaviour, this may produce two different modes of failure. In Sliding Shear Failure (Figure 4.7-b), the development of flexural cracking at the tense corners reduces the resisting section; failure is attained with sliding on a horizontal bed joint plane, usually located at one of the extremities of the pier. In Diagonal Cracking (Figure 4.7-c) failure is attained with the formation of a diagonal crack, which usually

DRAFT

develops at the centre of the pier and then propagates towards the corners. The crack may pass prevailingly through mortar joints (assuming the shape of a “stair-stepped” path in the case of a regular masonry pattern) or also through the blocks.



Rocking

Sliding Shear Failure

Diagonal Cracking

Figure 4.7. Typical failure modes of masonry piers.

The occurrence of different failure modes depends on several parameters: the geometry of the pier; the boundary conditions; the acting axial load; the mechanical characteristics of the masonry constituents (mortar, blocks and interfaces); the masonry geometrical characteristics (block aspect ratio, in-plane and cross-section masonry pattern). In previous studies, many experimental tests have attempted to analyse the influence of these parameters on the failure mode of masonry piers. In general, it has been argued that Rocking tends to prevail in slender piers, whereas Bed Joint Sliding is lightly to occur only in very squat piers. In moderately slender piers, Diagonal Cracking mode seems to prevail over Rocking and Bed Joint Sliding for increasing levels of vertical compression. Diagonal Cracking propagating through blocks tends to prevail over Diagonal Cracking propagating through mortar joints for increasing levels of vertical compression and for increasing ratios between mortar and block strengths. Increasing interlocking of blocks (block aspect ratio plus masonry pattern) may induce a transition from Diagonal Cracking through mortar joints to Rocking, to Diagonal Cracking through blocks or to Bed Joint Sliding. Crushing, in general, occurs for high levels of vertical compression (related to the compressive strength of the material).

It is worth noting that the classification of these failure modes is often implicitly referred to the pier element type. In fact, although a great effort has been devoted to study the behaviour of piers during the last decades (FEMA 306 and 307 collect some results of international experimental campaigns), yet tests on spandrels are very limited and quite recent. Indeed, the boundary conditions that characterize spandrel elements are very different from those of masonry piers (in particular due to the interlocking at the end-sections). As a consequence, the following differences may be noticed: in case of flexural behaviour, crushing represents a very rare instance owing to low values of axial load which characterize spandrel elements (especially with lack of tie-rods or r.c. beams); in case of shear behaviour, sliding failure (meant as sliding on a vertical bed joint plane at the end-sections), due to interlocking, is unlikely to happen.

DRAFT

Examples of the occurrence of aforementioned sub-classes of damage class A are presented in Figure 4.8-Figure 4.12.

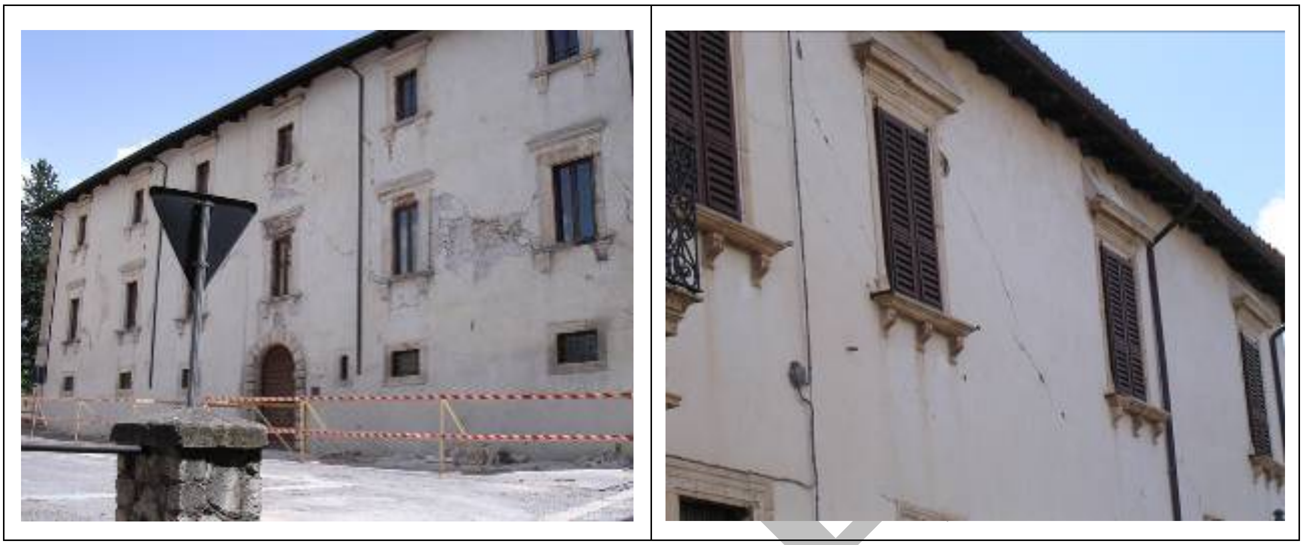


Figure 4.8. Class A – Damage to in-plane loaded walls: damage examples of sub-class A-a (cracks in piers).

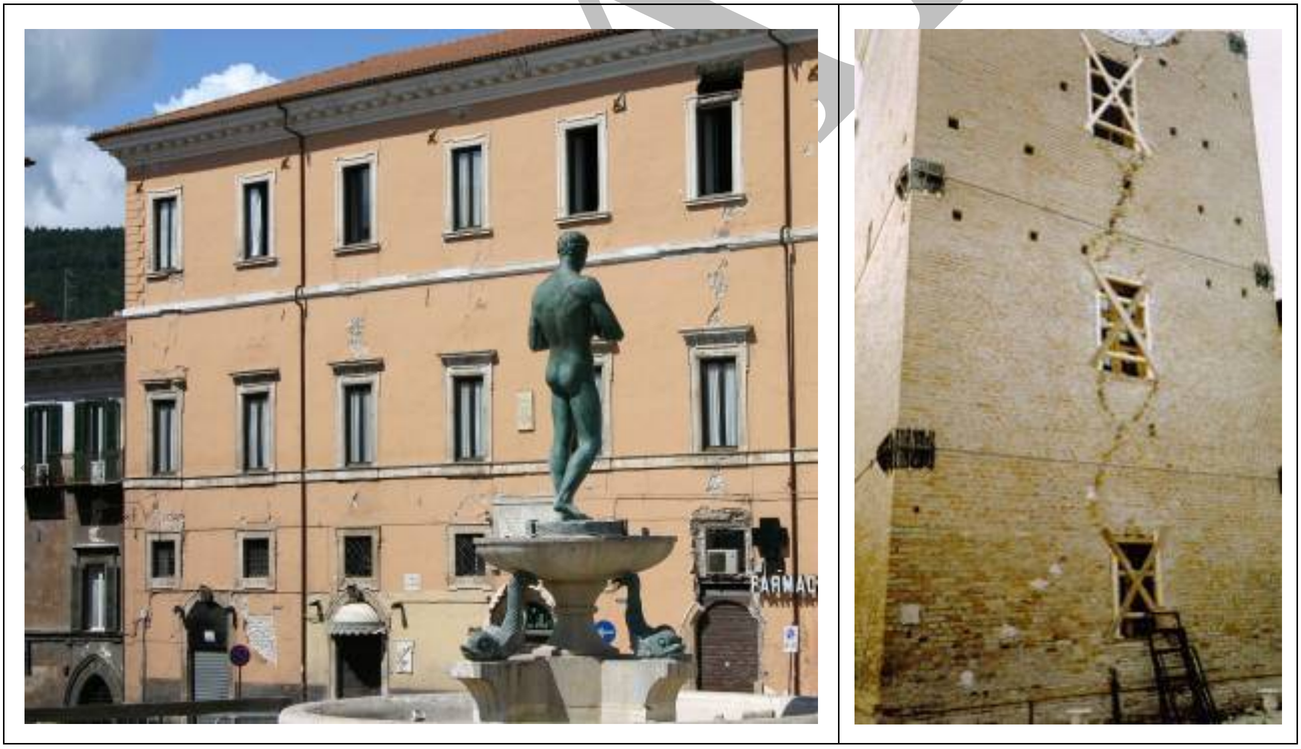


Figure 4.9. Class A – Damage to in-plane loaded walls: damage examples of sub-class A-b (cracks in spandrels).

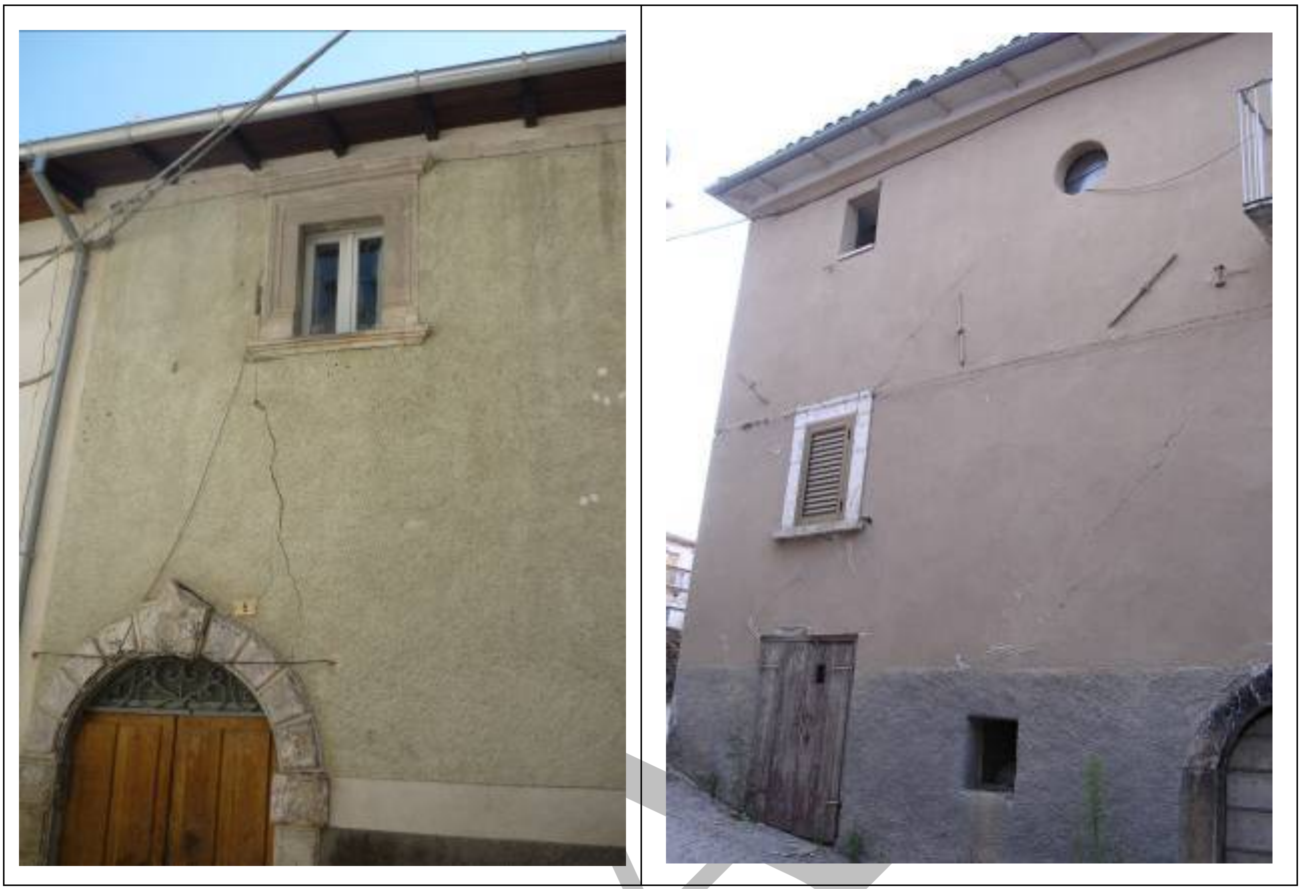


Figure 4.10. Class A – Damage to in-plane loaded walls: damage examples of sub-class A-c (generic diagonal cracking) in ordinary buildings.

DRAFT

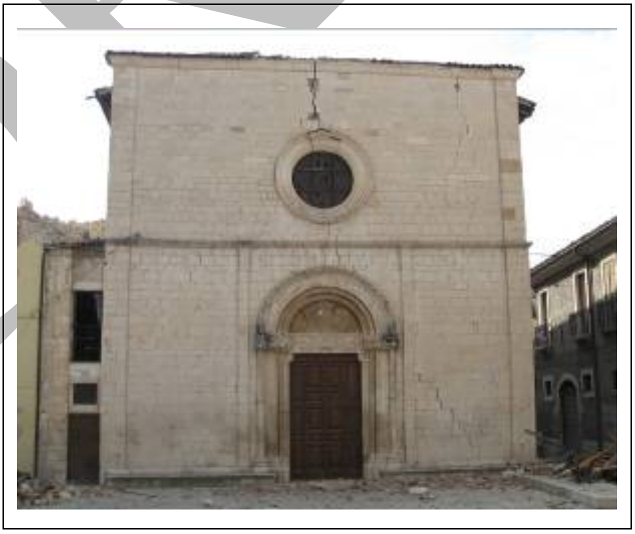


Figure 4.11. Class A – Damage to in-plane loaded walls: damage examples of sub-class A-c (generic diagonal cracking) in the façade of a church.

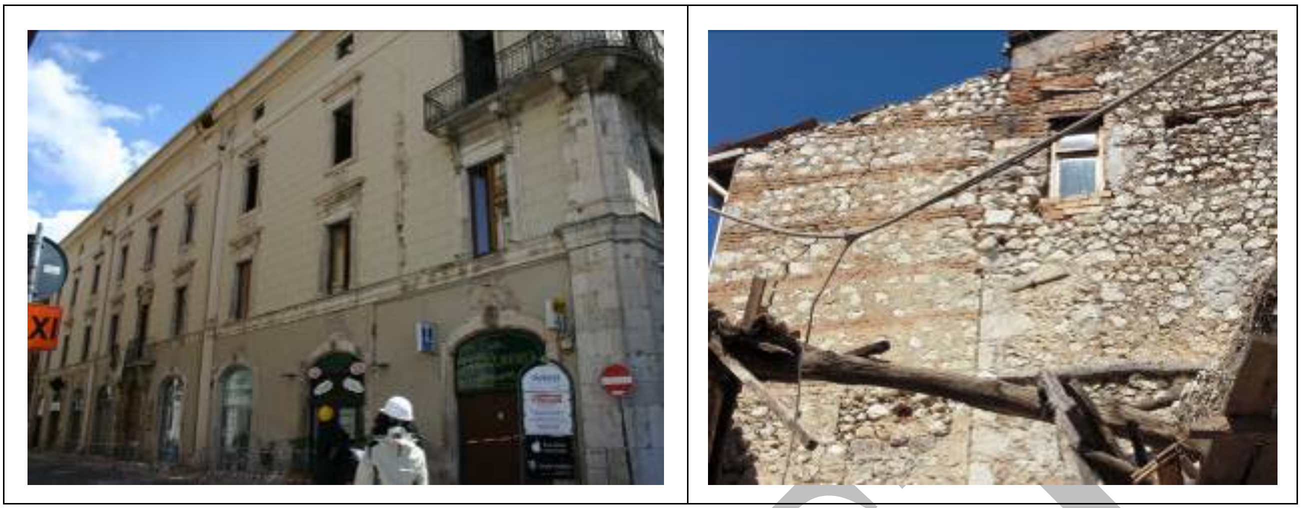


Figure 4.12. Class A – Damage to in-plane loaded walls: damage examples of sub-class A-d (cracks in structural gaps).

3.2.2 Class B - Damage to out-of-plane loaded walls

This class collects the out-of-plane damage and failure of masonry structures. These ones are often associated to the motion of single or multiple large masonry blocks; therefore, the collapse occurs due to loss of equilibrium. As a function of the number of blocks involved, two main sub-classes are defined:

- B-a: single-block overturning;
- B-b: multi-blocks overturning.

Each one of these sub-classes collects a wide range of possible motion phenomena as detailed in the following.

In case of single block overturning (B-a), the motion mainly involves the overturning of a single block of masonry, i.e. a single wall, a corner, etc…. The overturning may include the participation of the party walls (as illustrated in Figure 4.13 in cases 1 and 2) or not, depending on the connection between the involved walls. Frequently, in case of weak connections, the morphology of cracks is almost vertical, whereas, in case of quite good connections, diagonal cracks occur inducing the overturning of a portion of the perpendicular connected walls. The overturning may occur as a rotation around both horizontal hinge (Figure 4.13, cases 1, 2 and 4) or an inclined one (Figure 4.13, case 3). In some cases, the presence of vertical aligned openings in the main façade coupled with both a weak connection with the perpendicular walls and a weak interlocking at the interface between the end-sections of the spandrel and the adjacent masonry, may promote the activation of motions like the one illustrated in Figure 4.13, case 4. With reference to the participation of party walls, factors which mainly affect the connection quality are either different constructive phases of walls involved in the motion (in some cases, the connection is absent at all) or presence of openings at proximity of the corner.

DRAFT

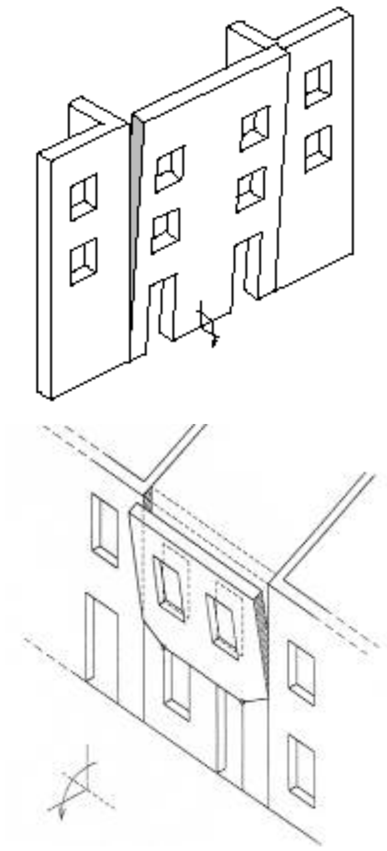
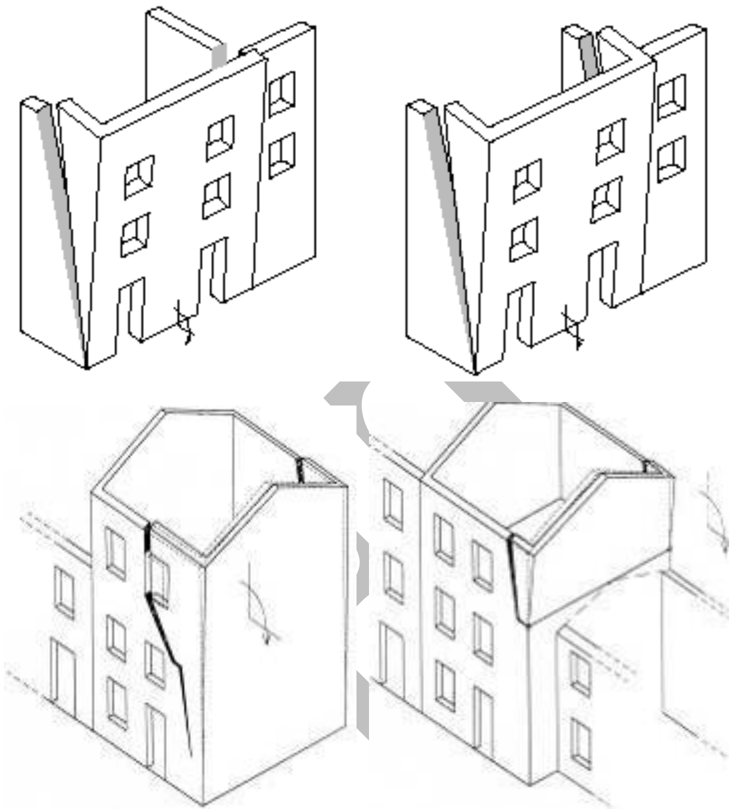
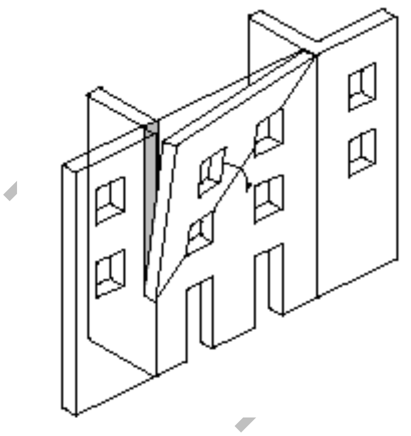
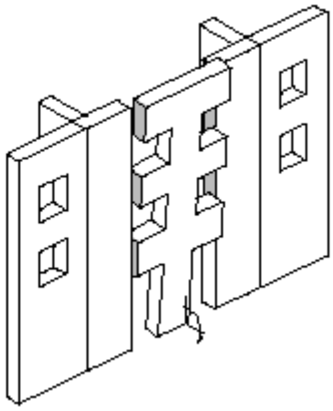
	
1. Overturning of a single wall in case of weak connection with party walls (rotation around a horizontal hinge).	2. Overturning of a single wall in case of good connection with party walls (rotation around a horizontal hinge).
	
3. Overturning of a single wall in case of weak connection with party walls (rotation around an inclined hinge).	4. Overturning of a single wall in case of weak connection with party walls (rotation around a horizontal hinge).

Figure 4.13. Class B: Damage to out-of-plane loaded walls; examples of damage belonging to sub-class B-a.

In case of corner overturning, the damage mechanism develops through the formation of diagonal cracks in the two walls involved. In general, it is quite simple to establish if the mechanism occurs by rotation around the corner or by two diagonal cracks that move inward from it. Also in this case, the motion may involve larger or smaller portions; in particular, the case in which overturning involves only portions located below the roof is a common one (in most cases due to the thrust of roof beams). Some sketches aimed to illustrated these damage modes are shown in Figure 4.14.

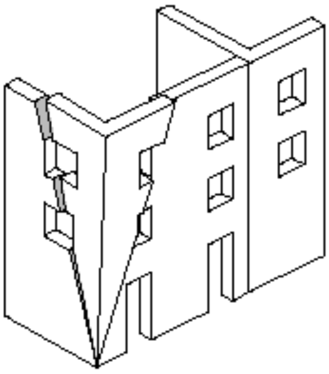
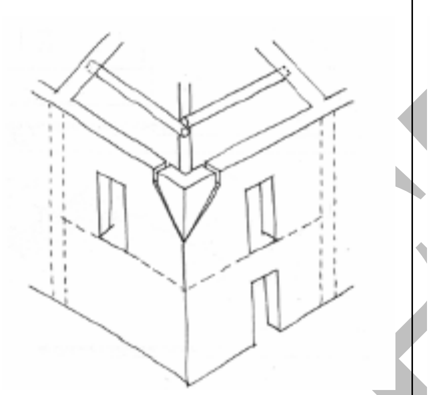
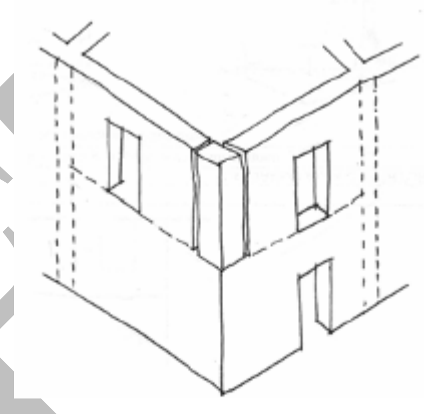
		
1. Corner overturning which involve a wide portion of two walls.	2. Corner overturning located at top portions.	3. Corner overturning which mainly involve the corner ejection.

Figure 4.14. Class B – Damage to out-of-plane loaded walls: examples of sub-class B-a involving the overturning of corners (sketch 1 has been carried from D'Ayala and Speranza 2003).

Since seismic response of an existing building is a quite complex mechanism, mixed behaviours, involving both the in-plane and out-of-plane response of walls, may occur. In some cases, the formation of diagonal cracks in the walls of the second direction, owing to the activation of their in-plane response, may contribute to promote a preferential path for cracks associated to the overturning of the façade. Indeed, often, in case of this type of mixed damage modes it is quite difficult to clearly identify the behaviour type that occurred at first. In general, it may be assumed that in class B, both purely out-of-plane and mixed in-plane/out-of-plane motions of masonry blocks are considered.

In case of multi-blocks overturning (B-b), we may distinguish between the case in which the top edges of blocks involved are free and that where the top edges are fixed. The related motions are quoted by some authors as "horizontal arch" and "vertical arch" mechanisms, respectively. Also in these cases, motions may develop around both horizontal or inclined hinges. Figure 4.15 and Figure 4.16 show some examples.

DRAFT

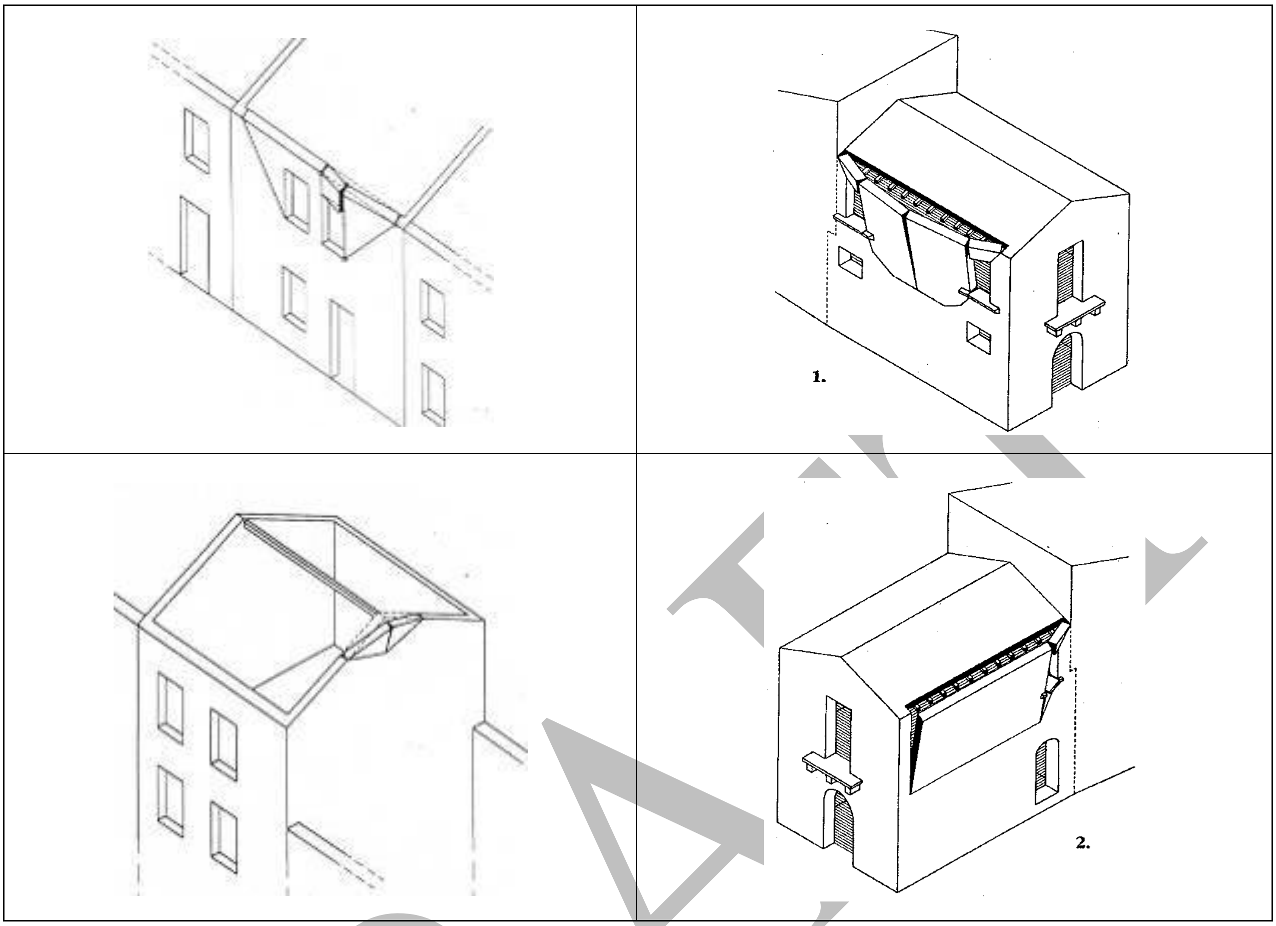


Figure 4.15. Class B – Damage to out-of-plane loaded walls: examples of sub-class B-b in case of top edge free.

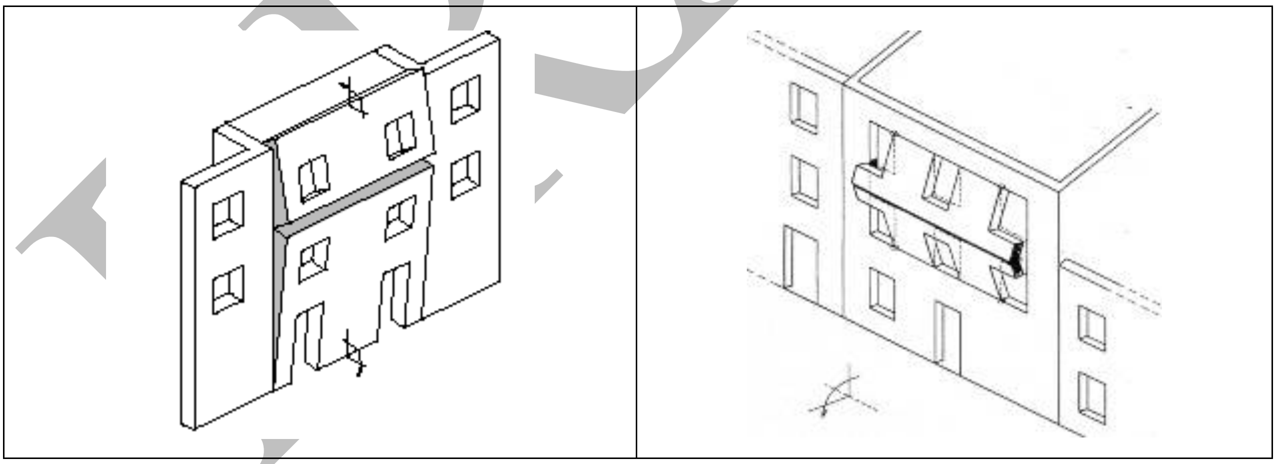


Figure 4.16. Class B – Damage to out-of-plane loaded walls: examples of sub-class B-b in case of top edge fixed.

In general, it is possible to notice factors that can affect the activation of overturning motion (both of single or multi-blocks): weak connection among masonry walls and floors; weak connection among masonry walls; presence of thrusting actions (e.g. owing to structural elements of roof or floor, vaults, etc…); irregularities in plan or elevation and presence of anti-seismic devices (as the presence of tie-rods, buttresses, etc…).

DRAFT

Figures Figure 4.17-Figure 4.22 aim to illustrate some examples related to the occurrence of the main sub-classes identified in damage class B.

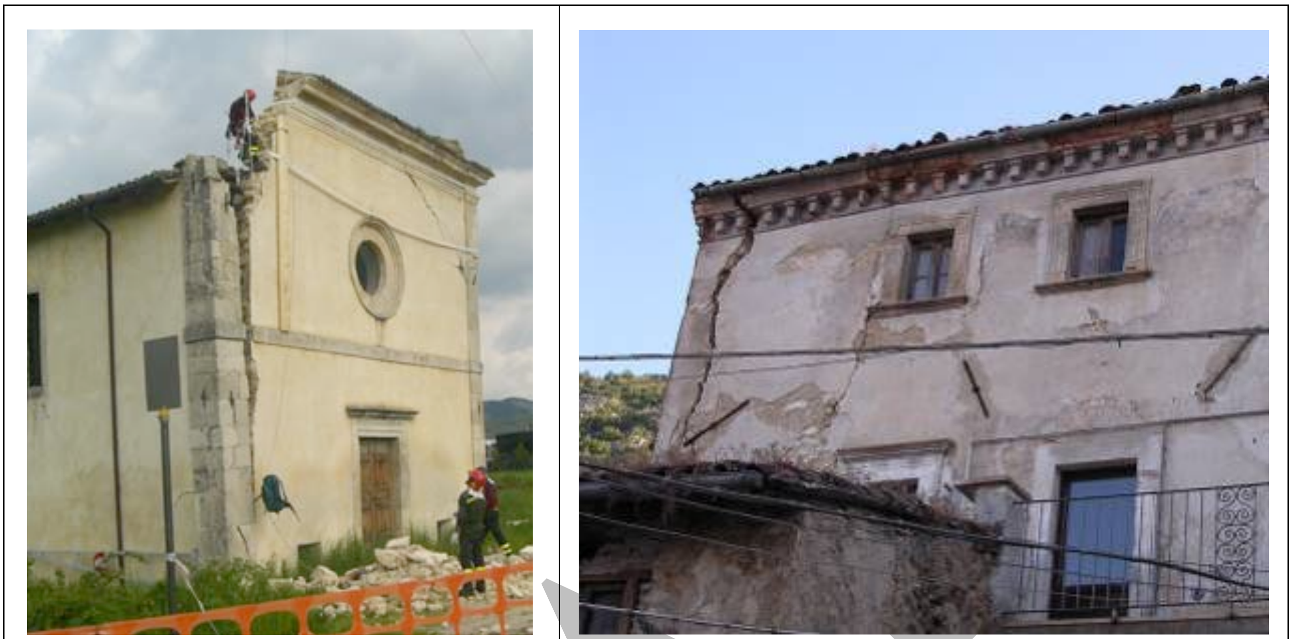


Figure 4.17. Class B – Damage to out-of-plane loaded walls: examples of sub-class B-a (motions which involve the overturning of a single wall).

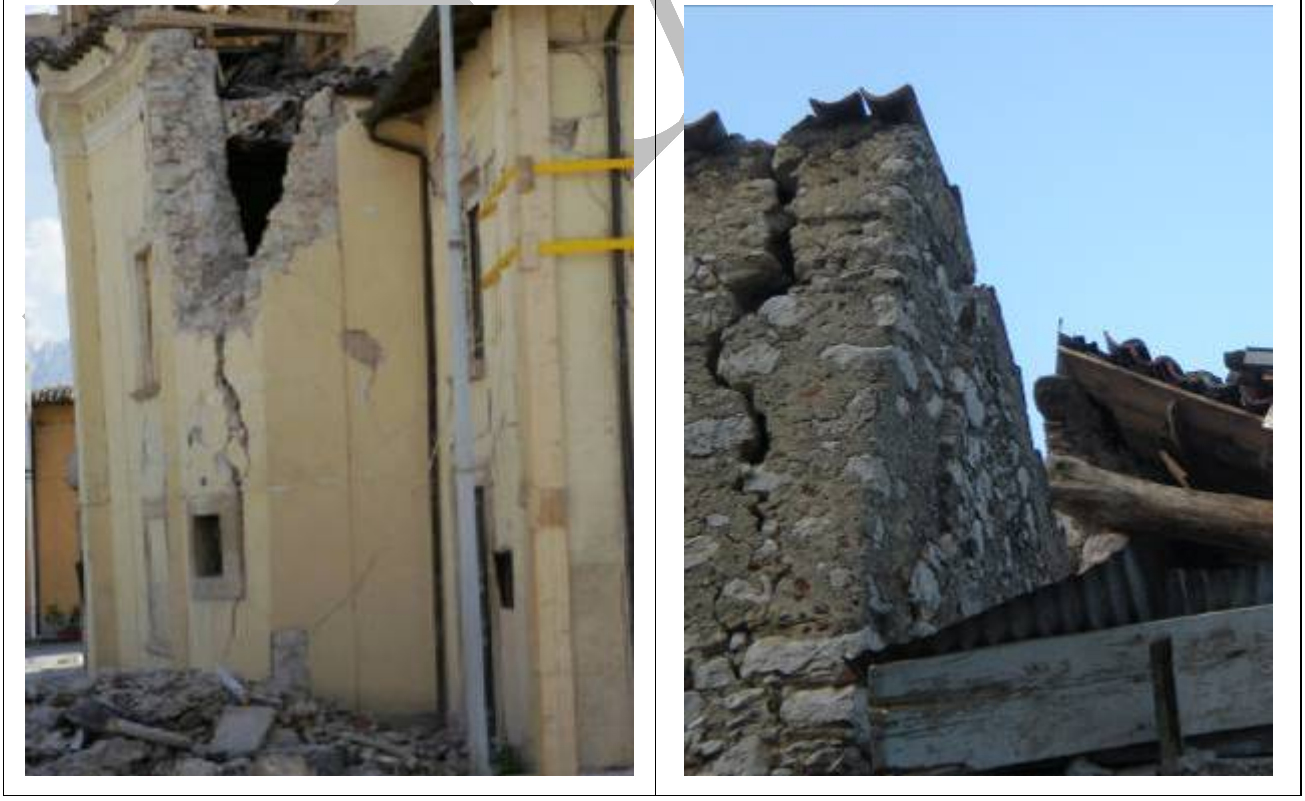


Figure 4.18. Class B - Damage to out-of-plane loaded walls: example of sub-class B-a (motions which involve the overturning of a corner).

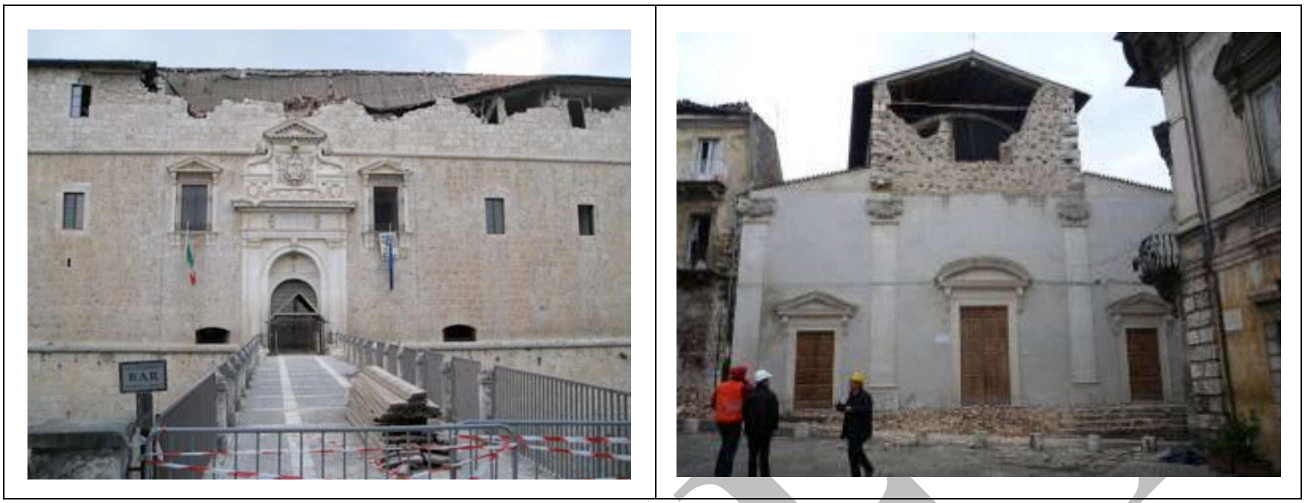


Figure 4.19. Class B - Damage to out-of-plane loaded walls: example of sub-class B-b (case in which the top edge of blocks involved is free).

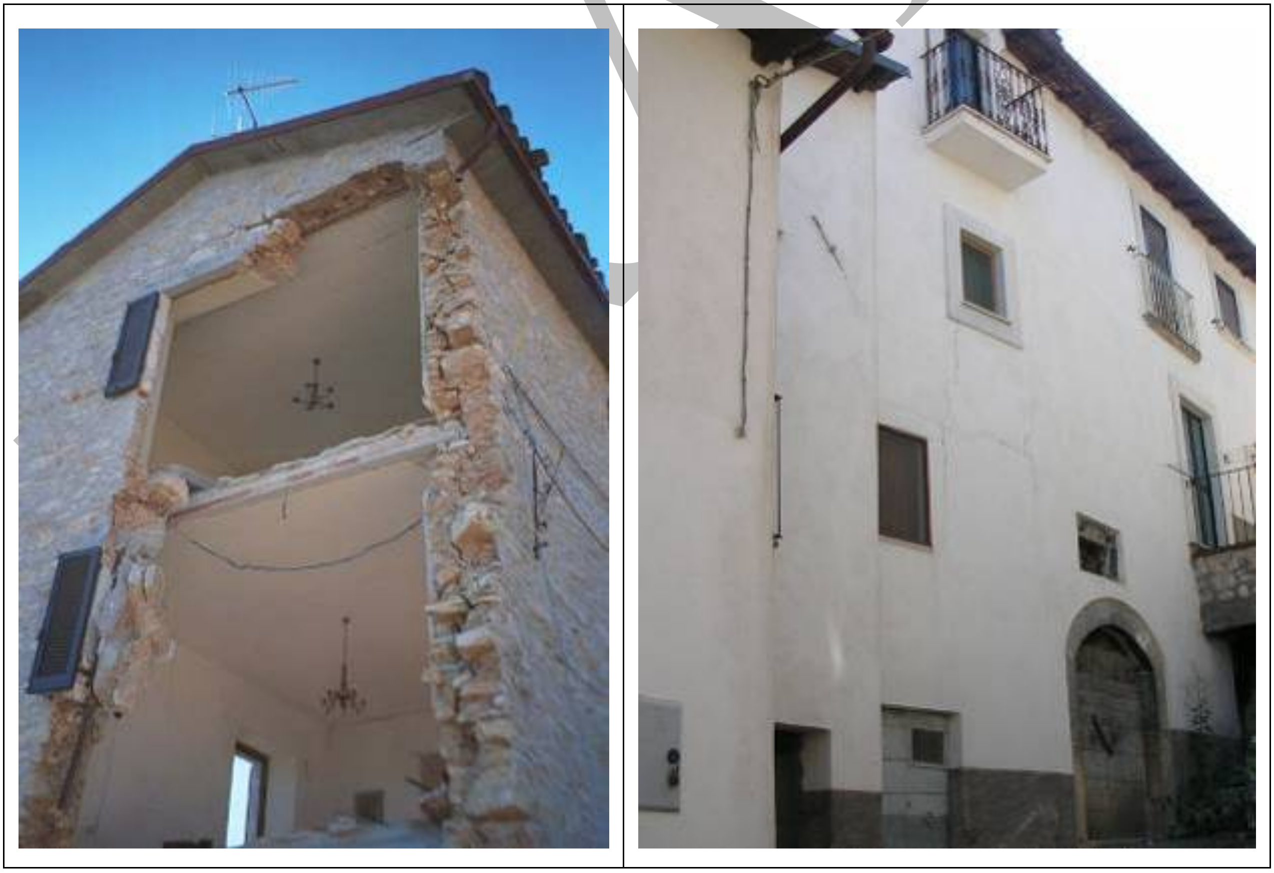


Figure 4.20. Class B - Damage to out-of-plane loaded walls: example of sub-class B-b (case in which the top edge of blocks involved is fixed).

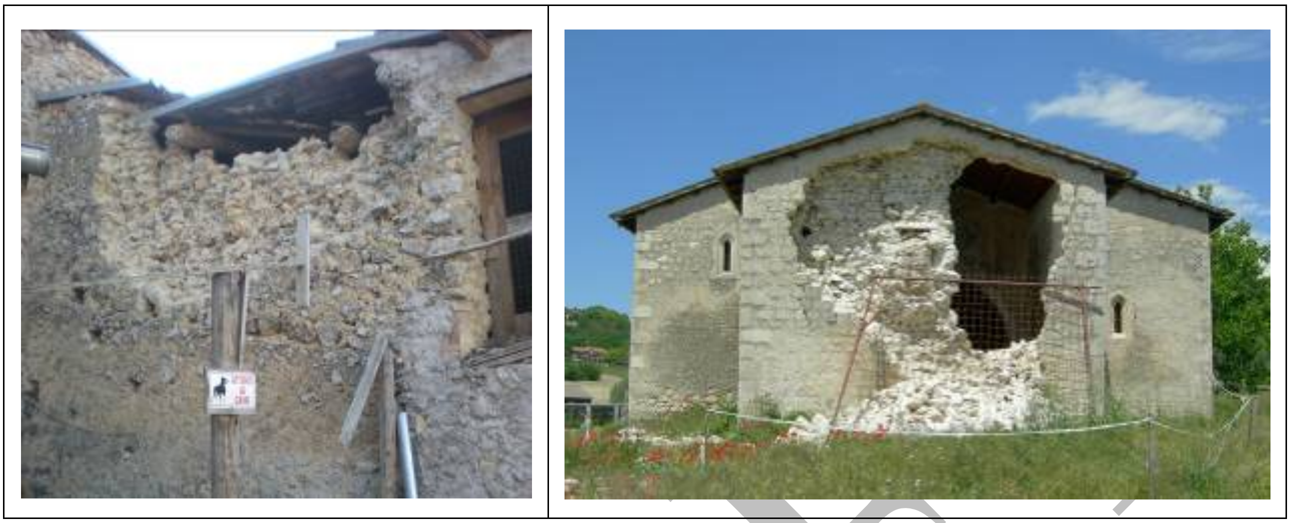


Figure 4.21. Class B - Effect on damage morphology due to the masonry quality.

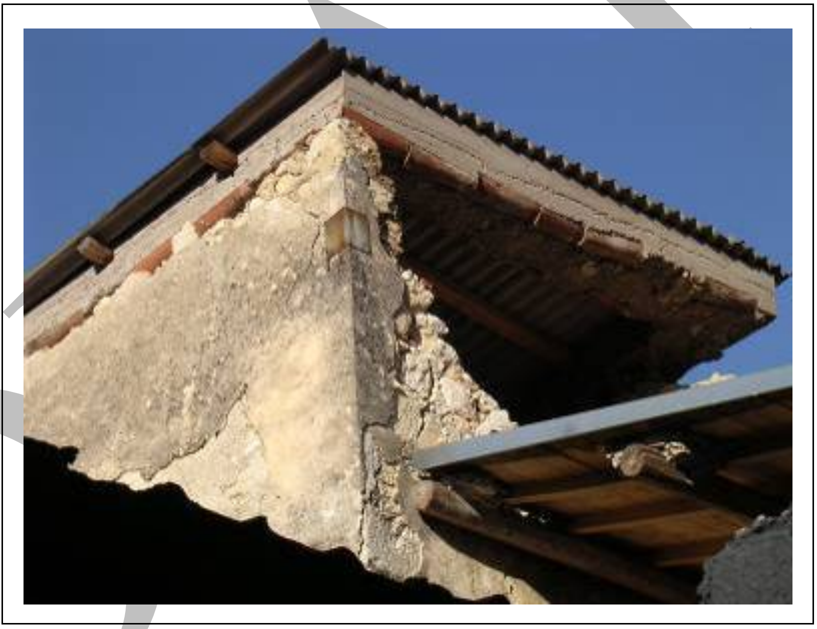


Figure 4.22. Class B - Example of possible effect due to the interaction between a heavy and rigid floor and an irregular masonry.

3.2.3 Class C - Damage to masonry elements under combined axial and bending loads

This class mainly refers to damage due to combined axial and bending loads. In particular, two main cases are considered: the case of whole slender structures (such as towers, bell-towers, …) and the case of single columns and pillars inserted in buildings. These structures are characterized by relevant slenderness and high compressive stresses on masonry (due to gravity loads, which may be increased in case of high vertical

DRAFT

components of the seismic action); for this reasons, they tend to fail in compression due to earthquake bending effects.

As previously discussed, in the case of flexural behaviour of masonry piers, two different modes of failure may occur as a function of the ratio between the applied vertical load and the compressive strength of masonry. If the applied vertical load is low with respect to the compressive strength, the horizontal load produces tensile flexural cracking at the corners and the element starts to behave as a nearly rigid body rotating about the toe (Rocking). If no significant flexural cracking occurs, due to a high applied vertical load, the element is progressively characterized by a widespread damage pattern, with sub-vertical cracks oriented towards the more compressed corners (Crushing). In both cases, the collapse is obtained by failure at the compressed corners. In general, in case of whole structures, the activation of bending effects is assessed by flexural cracking in correspondence of the more stressed transversal section. Usually, in case of isolated buildings, this latter is that at the base; however, but due to structural irregularities in elevation (presence of openings, reduction of wall thickness) it may be also at a higher level. In case of not isolated buildings (as an example, bell-towers placed beside a church), flexural cracking occurs at the level in which the structure become free. In case of columns/pillars inserted in buildings, damage is more frequently characterized by a widespread damage pattern with sub-vertical crack. Of course, as noticed in other classes, damage morphology may result in a quite different way as a function of the masonry type.

Figure 4.23 and Figure 4.24 aim to illustrate some examples related to the occurrence of damage class C.

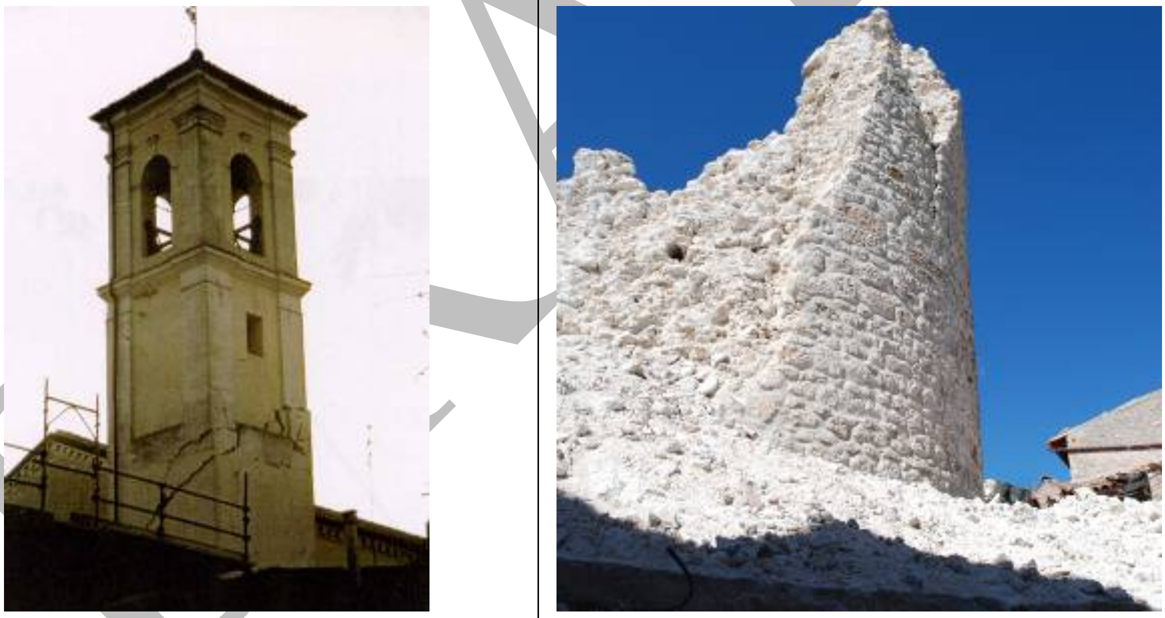


Figure 4.23. Class C - Damage to masonry elements subjected to combined axial and bending loads: towers.

DRAFT

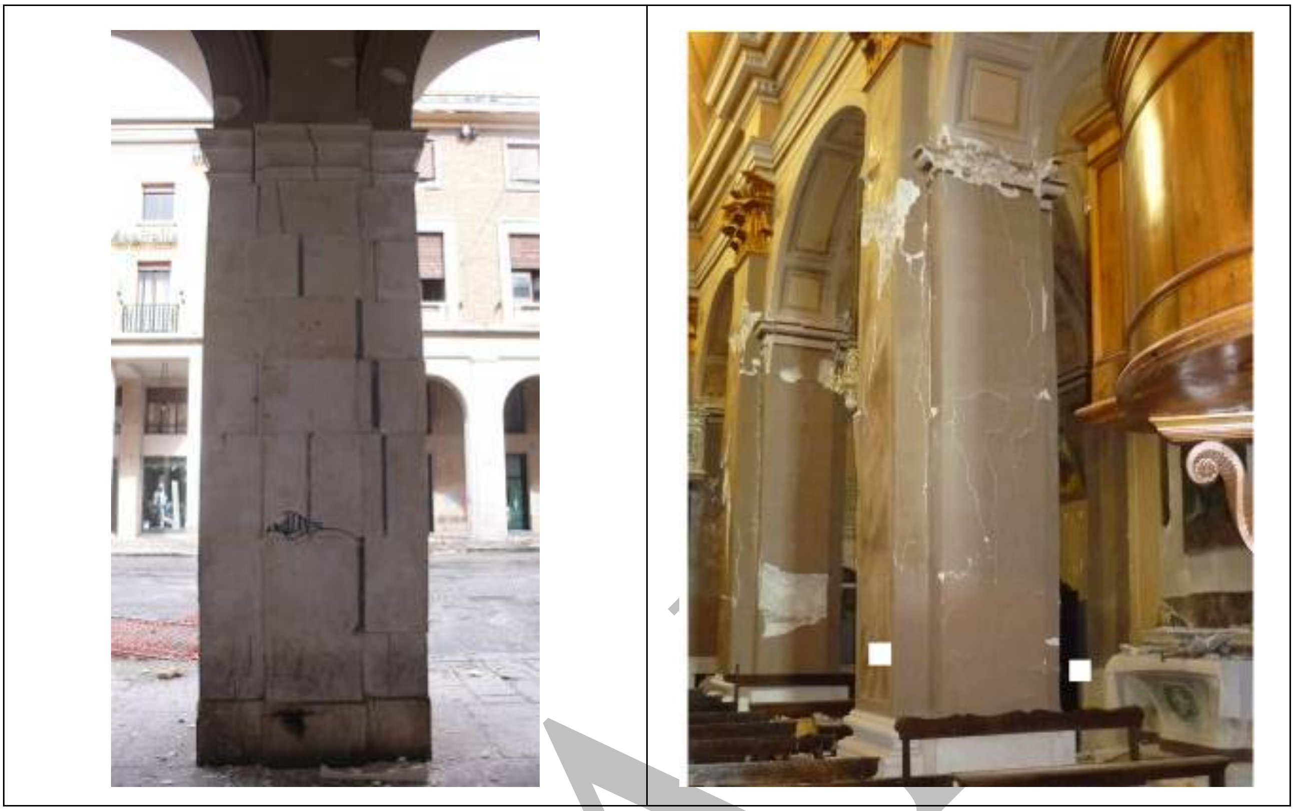


Figure 4.24. Class C – Damage to masonry elements subjected to combined axial and bending loads: single columns or pillars inserted in buildings.

3.2.4 Class D - Damage to in-plane loaded arches (or vaults)

In this class, arch structures loaded in their vertical plane are considered. Damage usually involves the arch-and-piers system and occurs by means of tensile cracks which tend to turn the structure into a set of blocks.

Collapse mechanisms are different from one structural typology to the other. Typically, the mechanism is represented by a four-hinge kinematism. The location of the hinges may change as a function of the slenderness of pillars and the geometry of arch and the presence of external forces (applied masses, external thrusts, force of tie-rods, …).

DRAFT

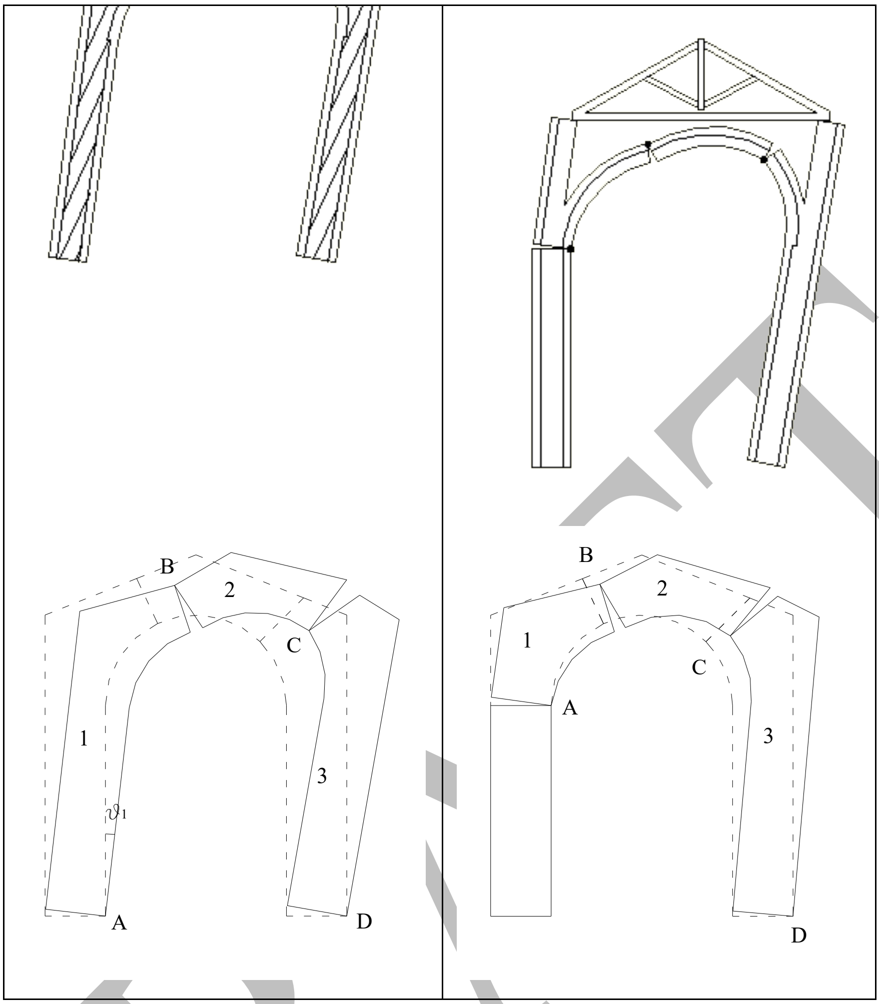
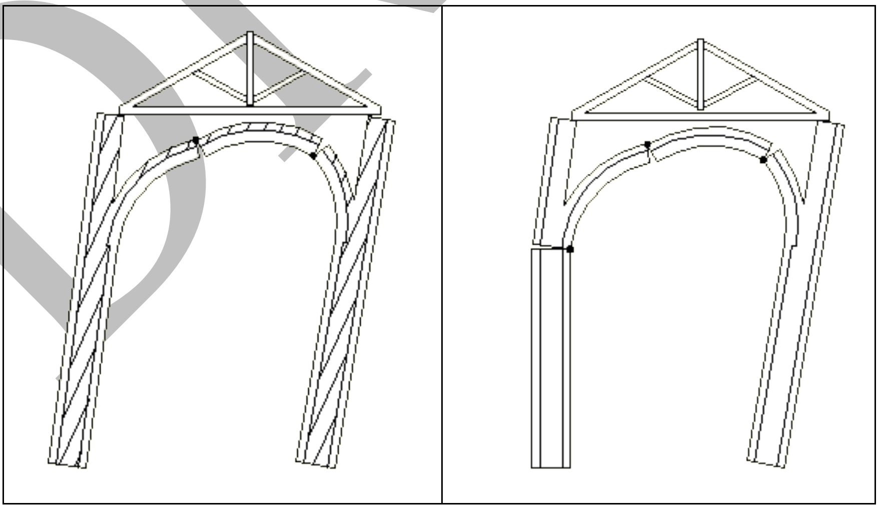


Figure 4.25 shows different possible response of the arch-pillar system typical of single-nave churches.



DRAFT

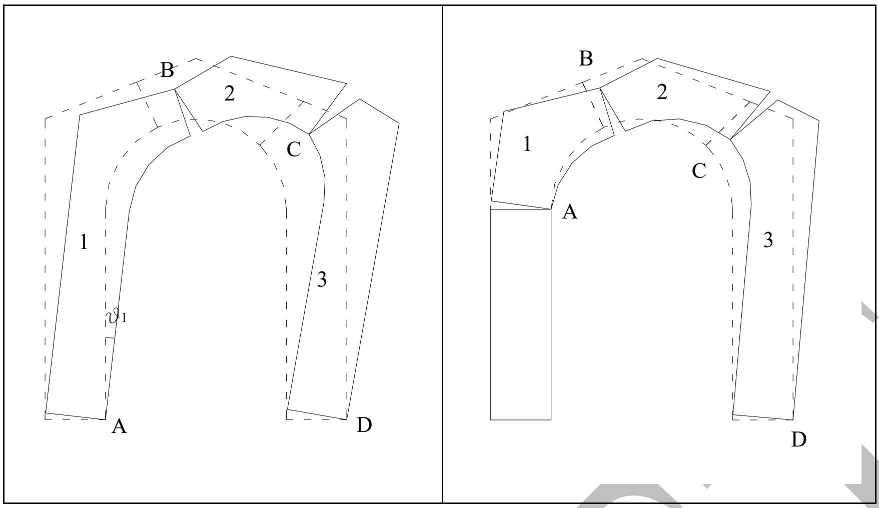


Figure 4.25. Class D – Damage to arched systems: possible mechanisms of the single-nave arch-pillars system.

It is worth noting how the presence of tie-rod (very frequent in case of arch structural system, both in the practice rules of construction and as retrofitting intervention) may significant affect the activation of this type of mechanisms. Despite this, it is important stressing how its effectiveness is related (in addition to the quality of the anchorages) to its internal pre-stress and to its position in the structure.

Figure 4.26 and Figure 4.27 shows some examples of this type of damage, for arches and vault, respectively.

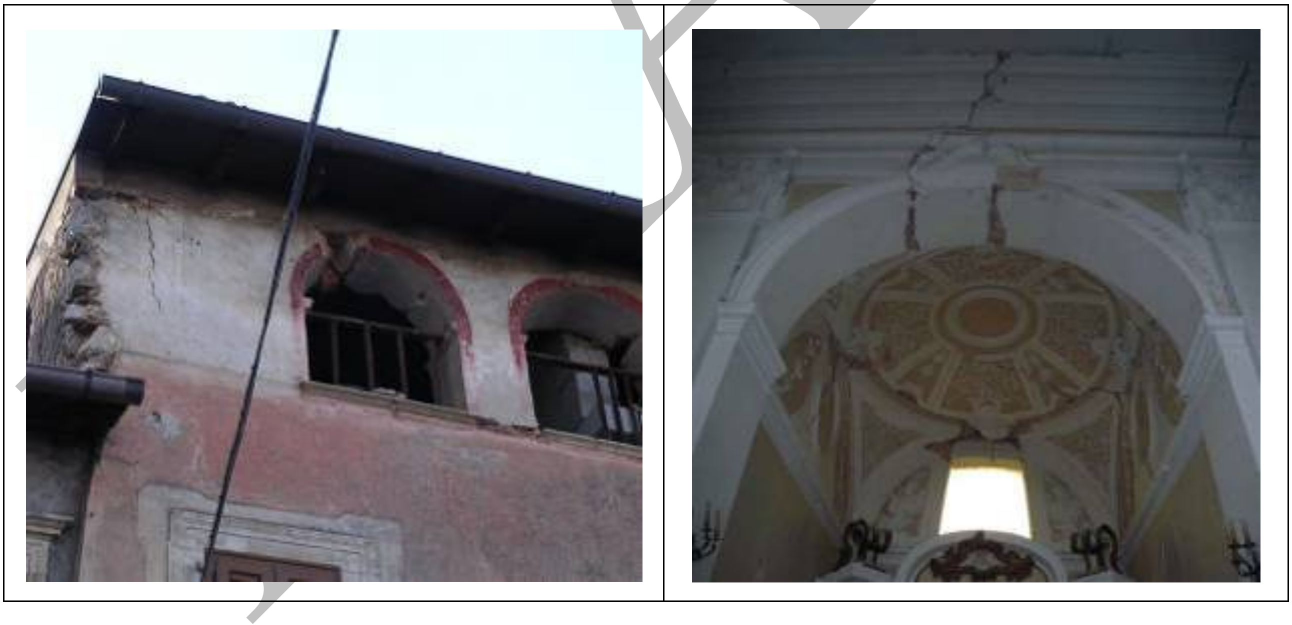


Figure 4.26. Class D –Damage to in-plane loaded arches.

DRAFT

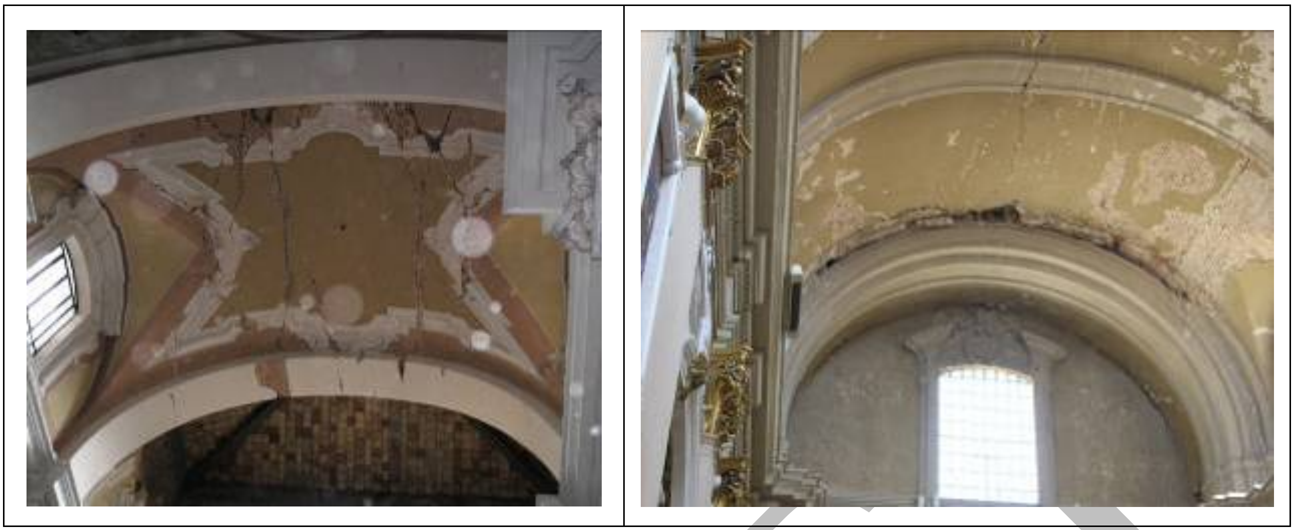


Figure 4.27. Class D – Damage to in-plane loaded vaults.

3.2.5 *Class E - Local damage of masonry*

This class mainly refers to damage restricted to single part of masonry continuum, which can be judged not so relevant to produce a significant decrease in load bearing capacity or failure of whole structural masonry elements. Different types of damage are included, which can be summarized in the following sub-classes:

- E-a: partial collapse of external leaf of masonry walls;
- E-b: damage to infill walls, cavities or structural gaps;
- E-c: local damage to masonry in correspondence of structural elements of floors/roofs or bolts of tie-rods.

Sub-class E-a includes both the detachment of external masonry leaf in multi-leaf walls and the spalling in massive masonry structures. The first case is particularly relevant in case of irregular masonry composed by not well connected multi-leafs, in which detachment is mainly associated to local instability phenomena. In the second case, cracks and spalling are substantially due to hydrostatic thrust effects.

Sub-class E-b mainly refers to cases in which damage occur in infill walls (as an example due to closure of openings) or cavities (as an example due to flue location). In the first case, in fact, since it quite difficult to re-establish a perfect interlocking between infill and adjacent masonry portions (in particular in case of irregular existing masonry), these parts may result as particularly exposed to damage.

Finally, sub-class E-c refers to local collapse or pounding of masonry due to the interaction with structural elements of floors/roofs (in particular in case of not well connected floors to masonry walls or vault thrusts) or bolt of tie-rods. The occurrence of this sub-class is particularly relevant in case of irregular masonry.

Figure 4.28 and Figure 4.29 shows some example of these sub-classes.

DRAFT

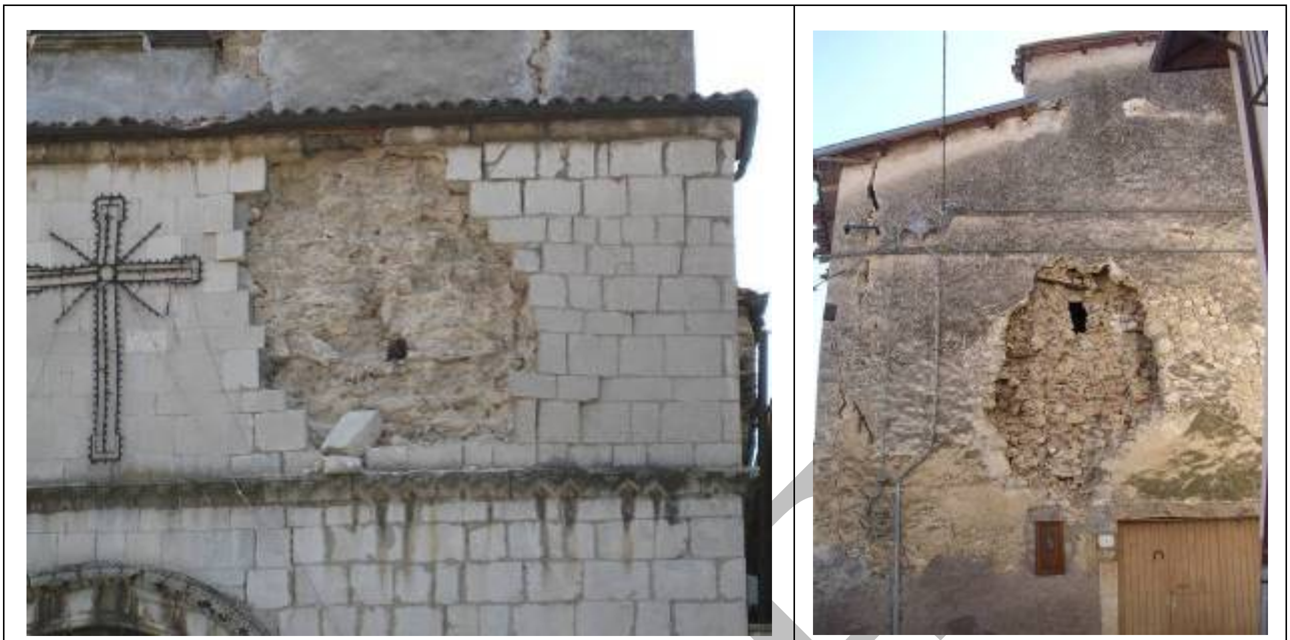
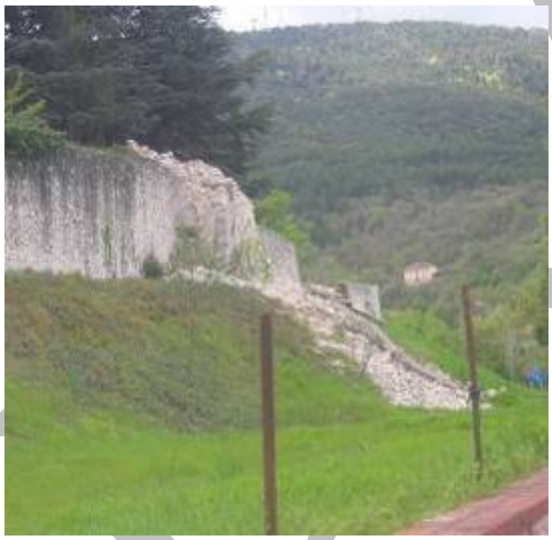
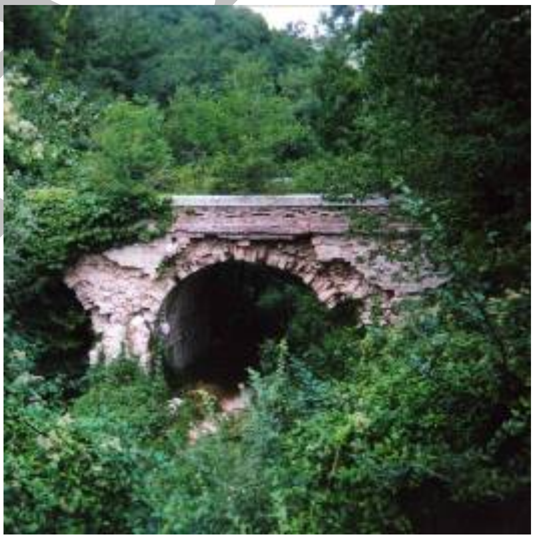


Figure 4.28. Class E – Detachment of external leafs in multi-leaf walls (E-a).

	
Spalling in surrounding walls (E-a).	Spalling in the spandrels of a bridge (E-a).

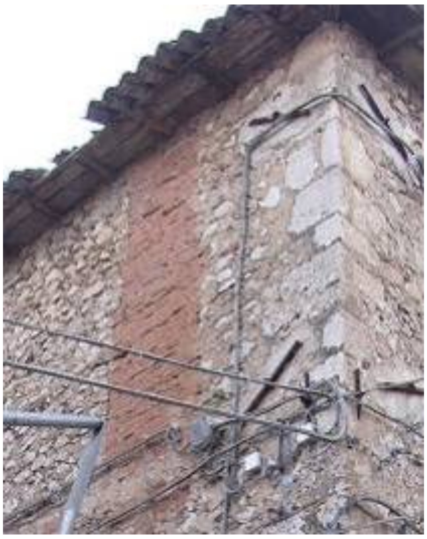
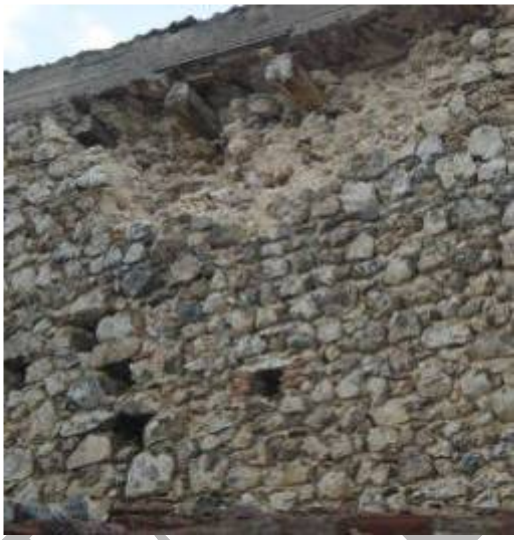
	
Damage in correspondence of the closure of an opening (E-b).	Collapse of masonry due to the interaction with structural elements of roofs (E-c).

Figure 4.29. Class E – Local damage of masonry

3.2.6 *Class F - Rocking of single or multiple blocks*

In this class, damage to self-standing elements or isolated columns are considered. Usually, due to their dimensions and boundary conditions, such elements are slightly compressed and tend to behave as single or multiple rocking blocks. In case of self standing elements, the connection to the main building plays a very crucial role; in many cases, rocking results in the complete failure of connection and in the full collapse of self-standing element. In case of columns made by large stone blocks (like as the case of Greek temples), sliding and rocking phenomena may interact. As highlighted by some authors, rocking mechanisms become more important when friction increases.

Errore. L'origine riferimento non è stata trovata. shows some examples of this damage class.

DRAFT

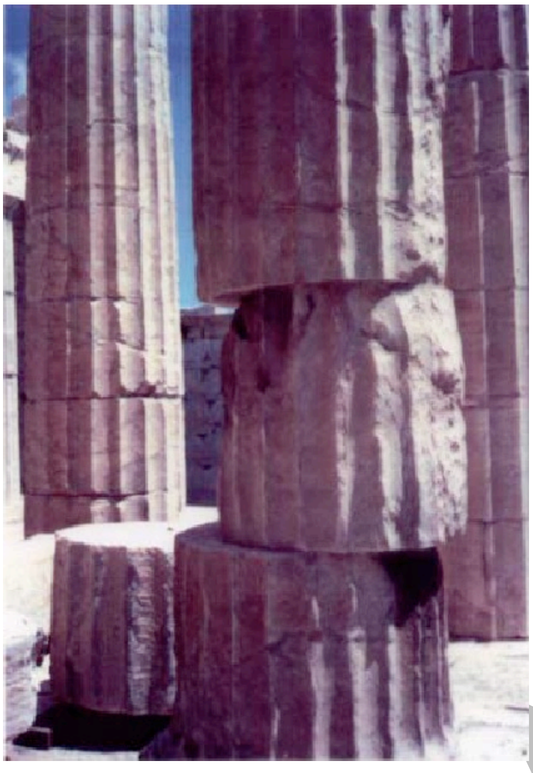
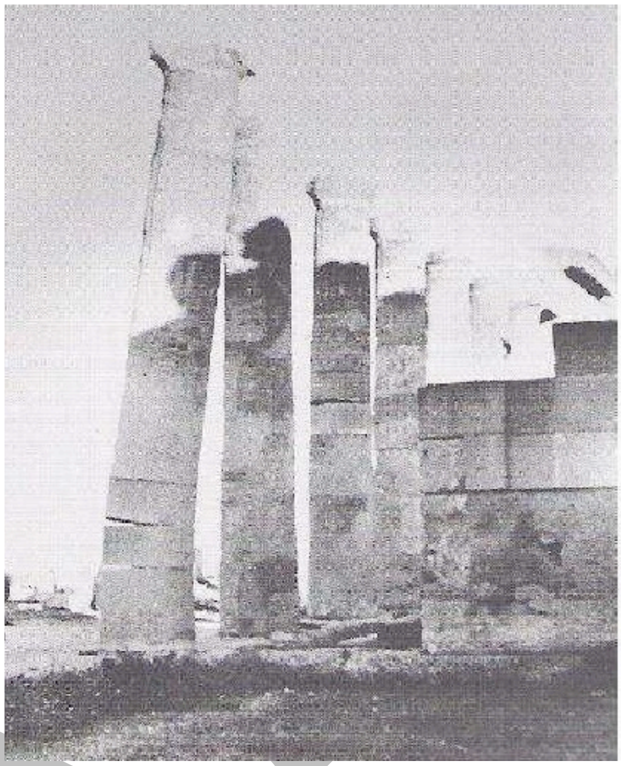
	
Response of isolated columns composed by large stone blocks: sliding occurred among adjacent blocks.	Response of isolated columns composed by large stone blocks: rocking with block failure.

Figure 4.30. Class F – Rocking of single or multiple blocks: case of columns made by large stone blocks.

3.2.7 Class G - Unthreading or disconnection of structural elements of roofs and floors

This class is specifically addressed to the damage which may occur in structural elements of floors and roofs (as an example, timber, steel or reinforced concrete elements).

In particular, the following sub-classes have been identified:

- G-a: damage in correspondence of supports of wooden/ steel floors and roofs;
- G-b: sliding of reinforced concrete beams in case of r.c. slab;
- G-c: damage to structural elements of floors/roofs.

In particular, in case of last sub-class, main reference is made to the unthreading of beams.

Figure 4.31 shows some example of this damage class. Of course, as a function of the damage level occurred, damage in structural elements and/or supports may result in the full collapse of floor/roof.

DRAFT

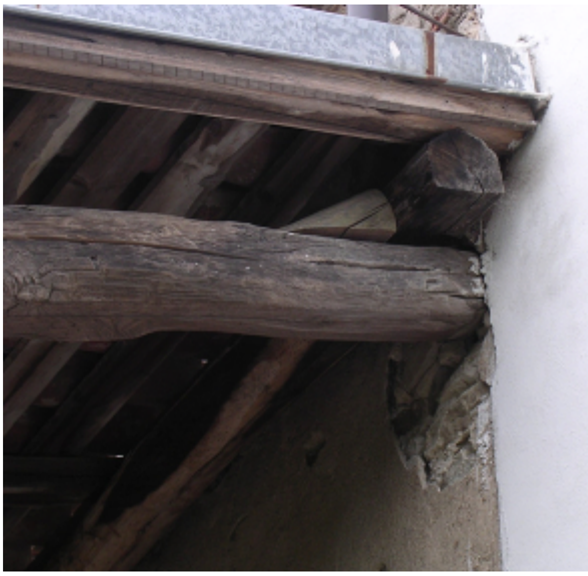
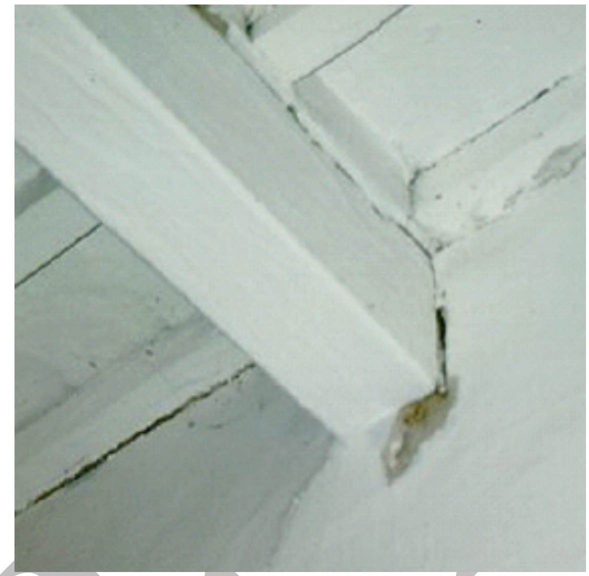
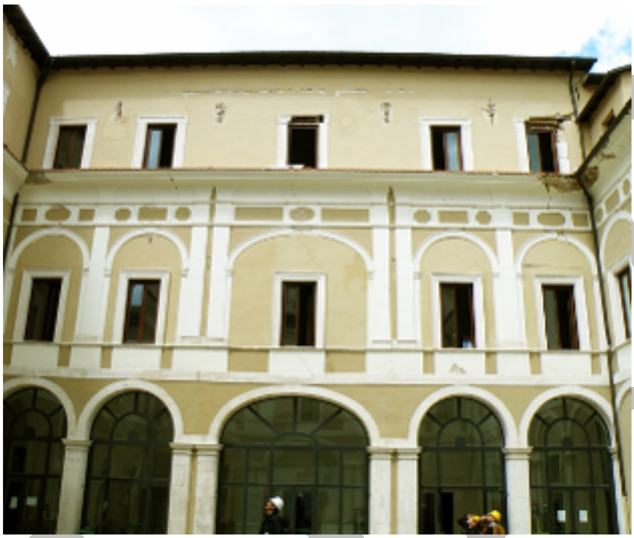
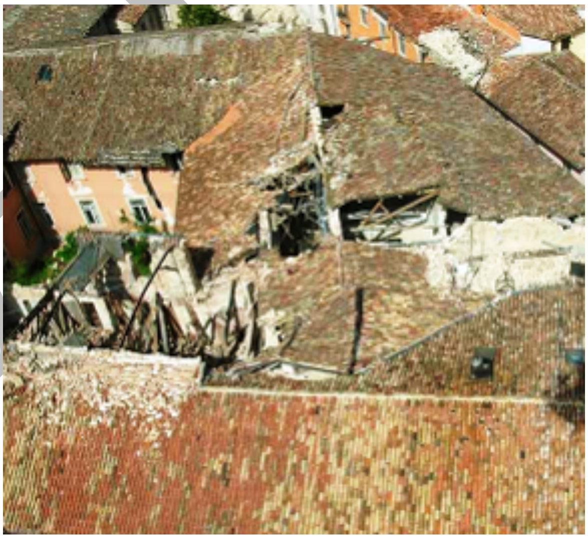
	
Sub-class G-a: unthreading of wooden beams.	Sub-class G-a: unthreading of wooden beams.
	
Sub-class G-b: sliding of r.c. beams.	Full collapse of a roof.

Figure 4.31. Class G – Unthreading or disconnection of structural elements of roofs and floors.

3.2.8 Class H - Drift of vaults or floors in their horizontal plane

Damage to vaults subjected to in-plane movements of their abutments are considered in this class. These movements usually produce shear cracks and local instability in vaults (particularly in case of thin vaults). Also in the case of floors, their capacity of transfer horizontal actions between walls is reduced with the increasing of the in-plane drift, due to damage.

Figure 4.32 shows some example of this class.

DRAFT

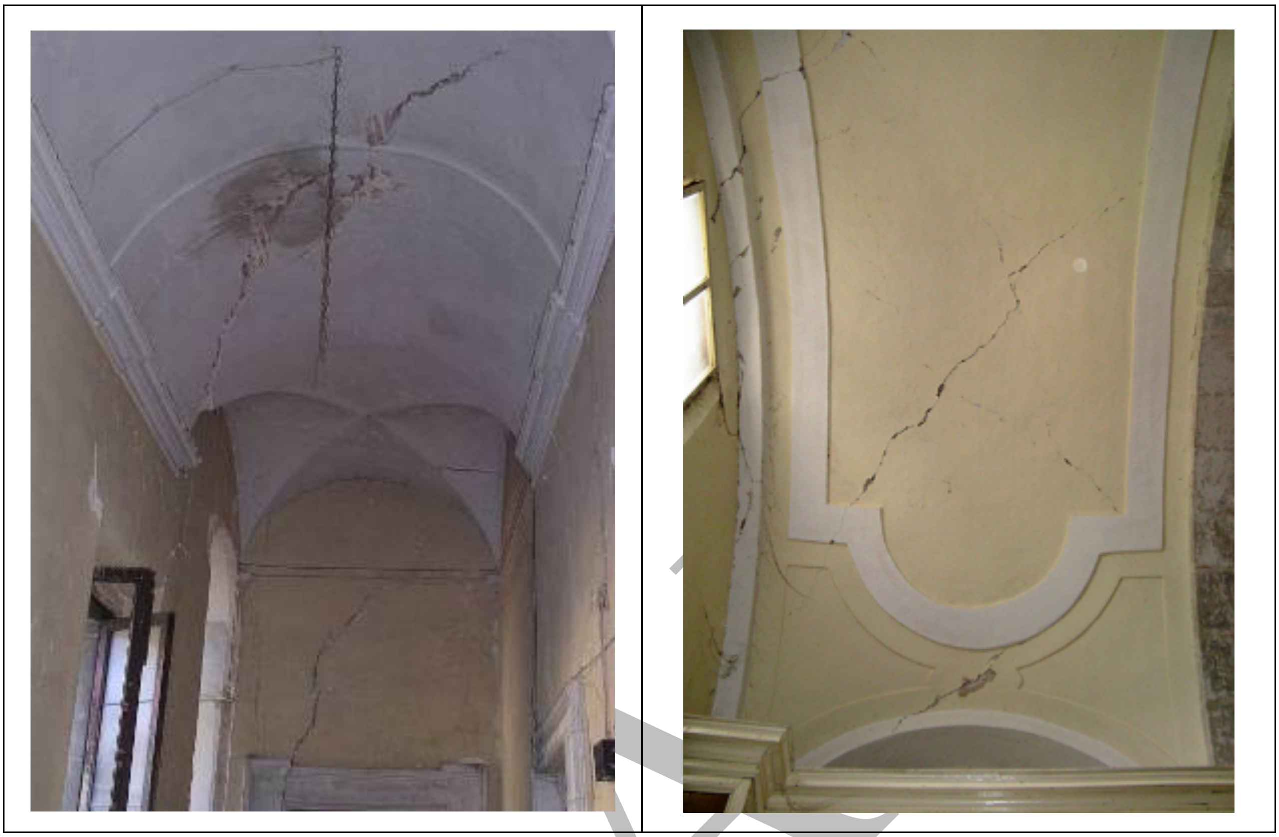


Figure 4.32. Class H – Lateral drift of vaults in their horizontal plane

3.2.9 Class I - Damage to domes

Typical three-dimensional damage is produced on domes due to their spatial configuration. Shear, out-of-plane and arch behaviours may coexist, producing complex damage states.

Figure 4.33 shows some examples of this damage class.

DRAFT

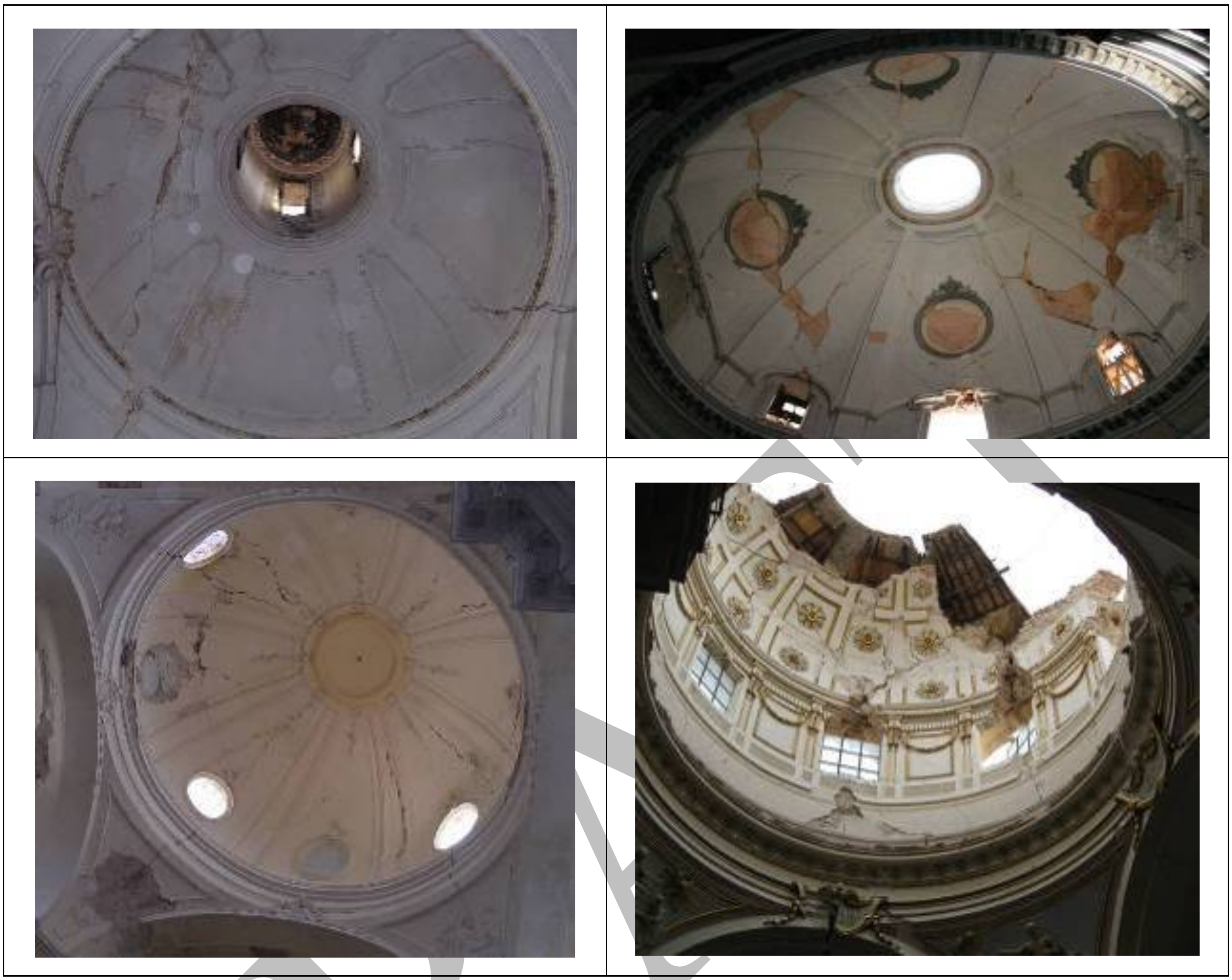


Figure 4.33. Class I – Damage to domes

DRAFT

3.2.10 Classification of architectonic assets

The need to classify architectonic assets derives from the observation that the occurrence of the different types of damage identified in the previous paragraph are strictly related to building morphology (architectural form, proportions) and technology (type of masonry, nature of horizontal diaphragms, effectiveness of wall-to-wall and floor-to-walls connections). These different behaviours call for different modelling approaches and different damage measures.

For this reason, the classification criteria here adopted is strictly “mechanical”. Thus, the different classes are defined on the basis of the most recurrent damage which they are subjected to. Since these latter are strictly related to building morphology, the classes of architectonic asset defined come to coincide with the main architectonic types of built heritage.

In Table 4.3 the classes are defined and a list of architectonic types related to each class is provided. Appendix A contains examples (pictures and images) of the assets belonging to each class. The classification should not be intended as rigid. As it will be showed in the following, single buildings may belong to different classes depending on their specific features.

Table 4.3. Classes and sub-classes of architectonic assets.

Architectonic asset class		Sub-class
A	Assets subjected to prevailing in-plane damage	A1 - palaces
		A2 - castles
		A3 - religious houses
		A4 - caravansaries
		A5 - collective buildings
B	Assets subjected to prevailing out-of-plane damage	B1 - churches
		B2 - mosques
		B3 - baptisteries
		B4 - mausoleums
		B5 - hammam
		B6 - modern theatres
		B7 - markets and bazaars
		B8 - industrial buildings
C	Assets damaged by high combined axial and bending loads	C1 - towers
		C2 - bell towers
		C3 - minarets
		C4 - lighthouses
		C5 - chimneys
D	Arched structures subject to in-plane damage	D1 - triumphal arches
		D2 - aqueducts
		D3 - bridges
		D4 - cloisters
E	Massive structures in which local failure of masonry prevails	E1 - fortresses
		E2 - defensive city walls
		E3 - Roman and Greek theatres
F	Blocky structures subjected to overturning	F1 - columns
		F2 - obelisks

DRAFT

		F3 - trilithes
		F4 - archaeological ruins
		F5 - Greek temples
G	Built systems subjected to complex damage	Historical centres

Buildings belonging to class A are typically characterized by a so-called "box behaviour". This means that the structure tends to behave as a whole and the collapse mainly derives from the in-plane damage of a number of masonry structural elements (refer to damage classes A-a and A-b). This type of behaviour may be attributed to those buildings made of vertical walls and horizontal diaphragms, with relatively good connections and relatively low amount of arched and vaulted systems. Vertical walls are usually rather slender (two dimensions prevailing on the third, the thickness) while diaphragms are enough stiff to convey seismic forces to masonry walls. In this types of buildings, inertial forces are transferred through diaphragms to masonry walls proportionally to their stiffness and strength. Since, due to geometry and limited tensile strength of the material, stiffness and strength of masonry walls subjected to in-plane or out-of-plane forces are very different (the in-plane loaded walls being more stiff), the walls parallel to the prevailing direction of the seismic force are subjected to higher seismic forces, acting in their plane. Thus, the collapse of the building derives from the collapse of these latter structural elements. Out-of-plane loaded walls, which are less resistant but carry lower seismic forces, may collapse for loss of equilibrium (refer to damage class B) only in case of particular lacks of the construction (excessive slenderness of a wall, lack of connection between perpendicular walls, presence of thrusting vaults).

Referring to the types of building which characterize the European building heritage, belong to class A:

- Palaces (A1), intended as relevant house or representative buildings (usually built from aristocratic families), made of at least two storeys. Their typical overall plan configuration is rectangular or L-shaped or C-shaped; one or more courtyards are often present. The space is usually characterized by rectangular units of limited width (rooms) defined by perpendicular systems of vertical walls and horizontal diaphragms made of wooden floors (sometimes replaced with steel or r.c. ones) or masonry vaults (usually placed at lower storeys). Figure 4.34 shows a typical example. In this framework, many exceptions should be obviously considered. For example, due to the representative role of this type of buildings, wider rooms with no intermediate diaphragms may be present (reception or party rooms, dancing rooms, libraries, …). The greater slenderness of the walls circumscribing these rooms (related to both lack of intermediate floors and lack of perpendicular walls), usually induce local out-of-plane damage. Moreover, in the courtyards or on the front face of the building, loggias or porticos, often characterized by arched structures, are present. Also these elements may be subjected to specific types of damage.

- Castles (A2), intended as military buildings or particularly relevant residential buildings built by royal or noble families. Similar to Palaces, they are usually isolated and often characterized by standing out elements used for defensive scopes (sighting towers). Figure 4.35 shows a typical example.

- Religious houses (A3), intended as buildings owned and inhabited by religious orders typical of Christian tradition (abbeys, convents and monasteries). These types of buildings are usually part of very complex built systems, where usually a church is also present. Except for common spaces, they are usually

DRAFT

characterized by rather small rooms. Arched cloisters and porticos are usually present as walking or communicating spaces. Figure 4.36 shows a typical example.

- Caravansaries (A4). Typical of North Africa and South-Eastern Europe, they are large inn enclosing a courtyard providing accommodation for caravans (Figure 4.37). Typically, these buildings have a square or rectangular walled exterior, with a single portal wide enough to permit large caravans to enter. The courtyard, almost always open to the sky, is often porched, and the inside walls of the enclosure are outfitted with a number of identical stalls and small rooms to accommodate merchants and their servants, animals, and merchandise.
- Collective buildings (A5). This sub-class of buildings collects all those buildings that have been built with collective functions, such as public houses, schools and universities, hospitals, museums…(Figure 4.38). From a morphological point of view, they are similar to palaces, since they are usually characterized by more than two storeys and modular systems of rooms. Often, linear distribution paths are present. Moreover, due to their public nature, they are usually characterized by some large spaces (stairs, entrance hall, meeting rooms) and by porticos and covered walking spaces.

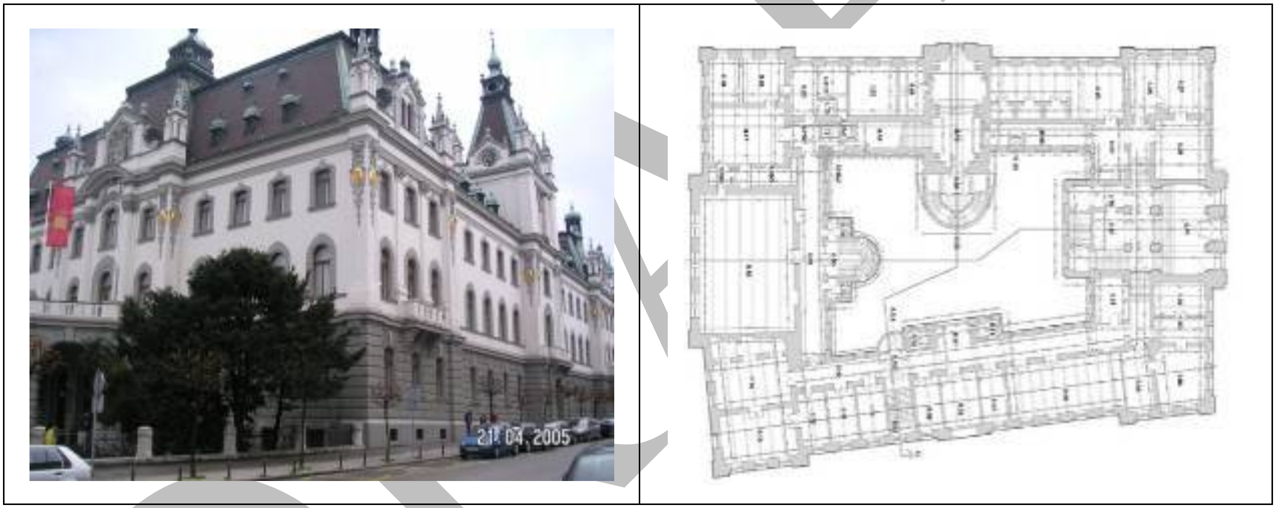


Figure 4.34. Class A1 – Palace Hassan Beys in Rhodes, Greece.

DRAFT

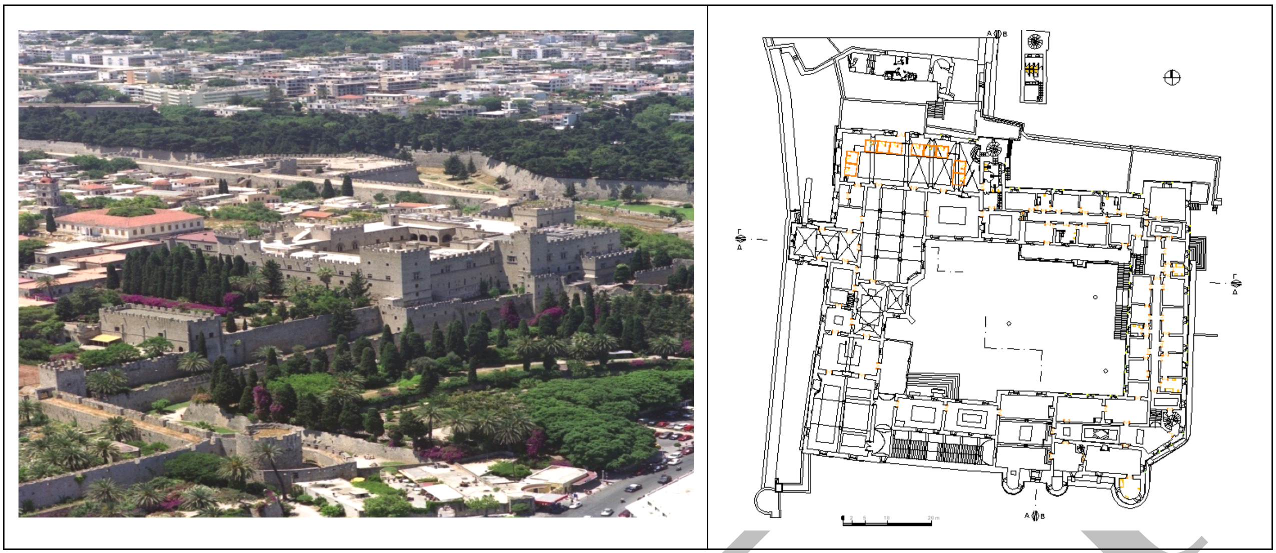


Figure 4.35. Class A2 – Great Masters Castles in Rhodes, Greece.

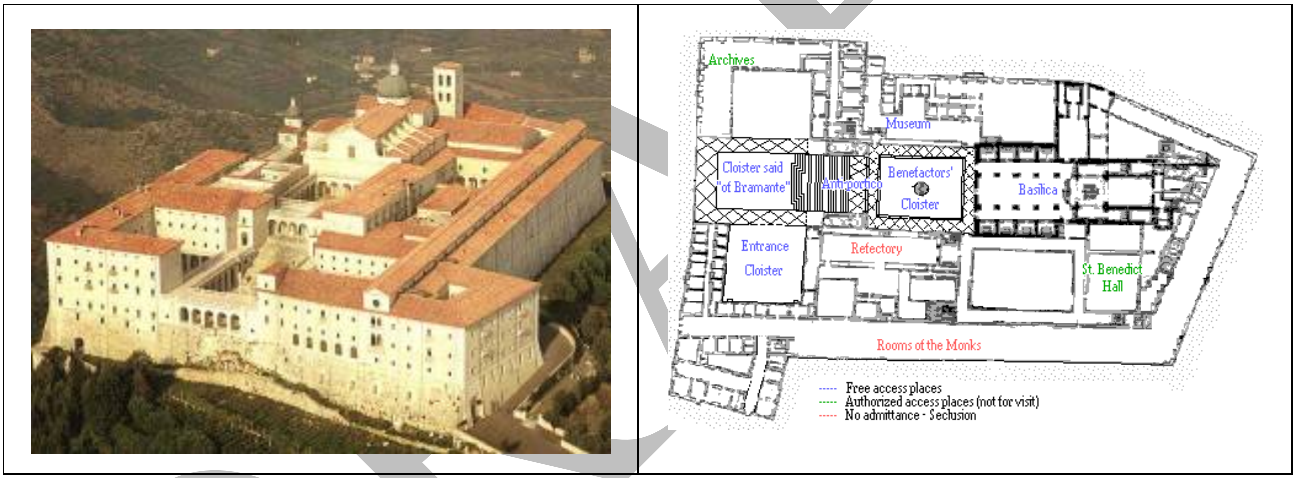

Archives
Museum
Cloister said "of Bramante"
Anti-portico
Benefactors' Cloister
Basilica
Entrance Cloister
Refectory
St. Benedict Hall
Rooms of the Monks
Free access places
Authorized access places (not for visit)
No admittance - Seclusion

Figure 4.36. Class A3 - The abbey of Monteccasino, Italy.

DRAFT

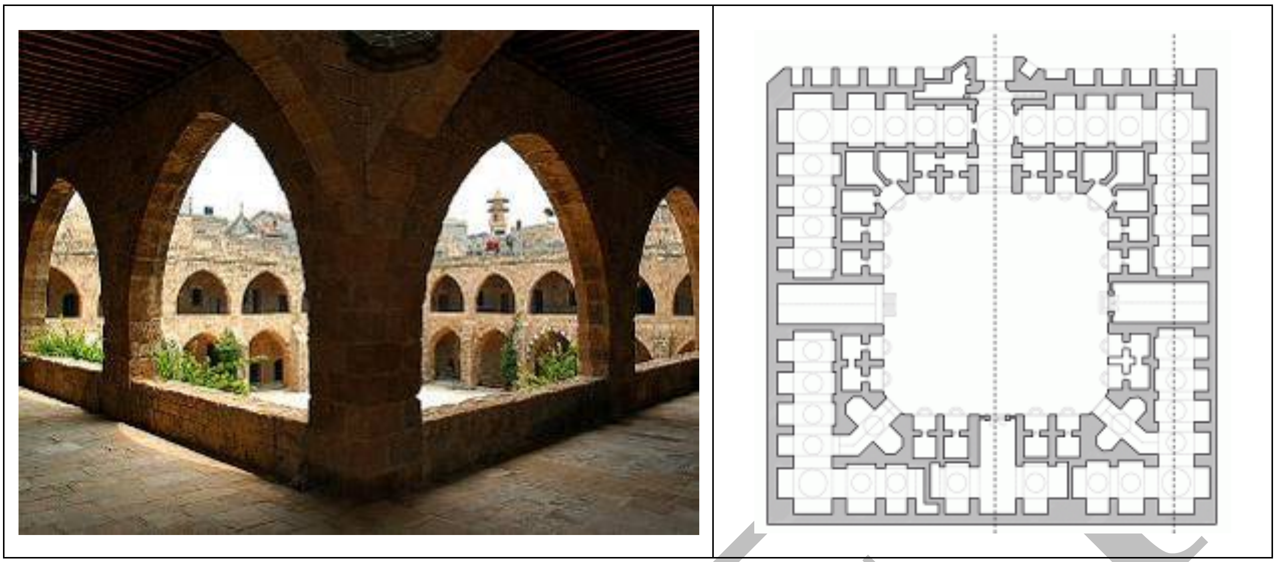


Figure 4.37. Class A4 - Khan El Frange, Lebanon.

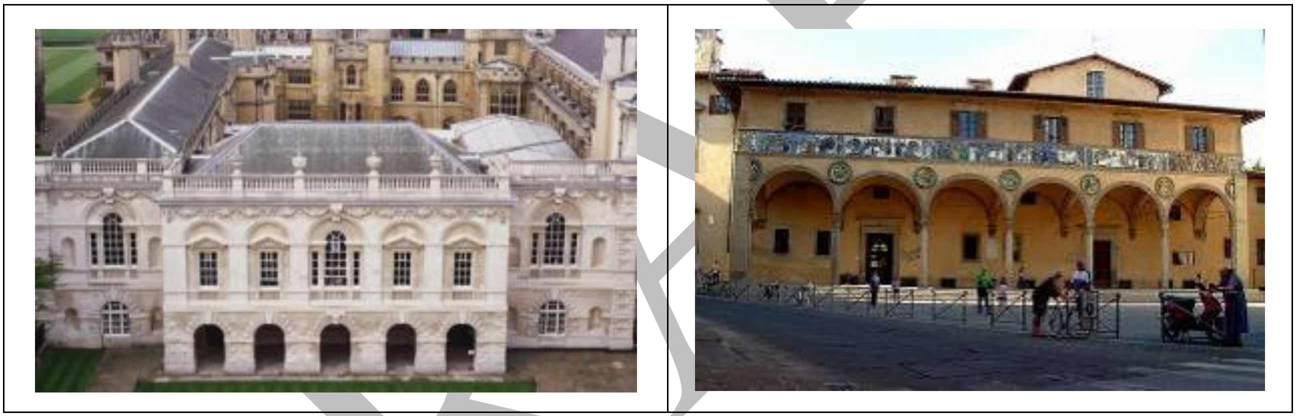


Figure 4.38. Class A5 - The University of Glasgow (UK) and the “Ospedale del Ceppo” in Pistoia (Italy).

In buildings belonging to class B out-of-plane damage are prevalent. This is typical of those buildings in which walls are particularly slender (owing to the absence of horizontal diaphragms) and in which connections between structural elements are weak (lack of tie-rods, scarce interlocking between walls,). In fact, in these cases, each wall of the building tends to behave independently from the other and is theoretically subjected to forces directly proportional to the seismic actions. The walls oriented perpendicularly to the prevailing acting forces tend to collapse for loss of equilibrium. This lead to a series of local failures, which may produce the overall collapse of the building. In European and Mediterranean built heritage, the buildings that more typically presents the above aforementioned constructive characteristics are religious ones (churches, mosques, baptisteries,). This is owing to their special function, requiring large and high open spaces for the collective rite. Besides them, also markets and theatres, in which more secular collective rites are carried on, present similar characteristics. Finally, masonry industrial buildings typical of XIX century should be considered, owing to their large spaces motivated by the presence of the production machines and engines. In synthesis, belong to class B the following types of buildings:

DRAFT

- Churches (B1) and Mosques (B2) (Figure 4.39 and Figure 4.40) and other religious buildings (B3 and B4). These are collective buildings in which the believer community meet to attend to religious ceremonies. Many types of buildings, very different for plan and elevation configuration, are comprised in this sub-class (roman basilicas, orthodox churches, protestant temples, mosques…). However, from the seismic point of view, they are all characterized by a common feature: the presence of a large hall/nave with slender lateral walls.

- Hamas (B5), intended as Turkish baths (Figure 4.41). They are made of series of quite large halls heated at different temperatures by a continuous flow of hot, dry air.

- Modern theatres (B6). These buildings are used for public theatrical or musical representations (Figure 4.42). They are typically characterized by a large entrance hall, a very large hall with a scene and large technical spaces (often very high due to the presence of illustrated curtains).

- Markets and bazaars (B7), intended as public spaces in which goods and merchandise are sell (Figure 4.43, on the left). They are usually covered or semi-covered large spaces made of serial structures (typically arches). They are often characterized by large passageways.

- Industrial buildings (B8). Reference is made to those masonry factories built between XVII and XIX century (Figure 4.43, on the right), characterized by large and long spaces where production machines and engines were placed.

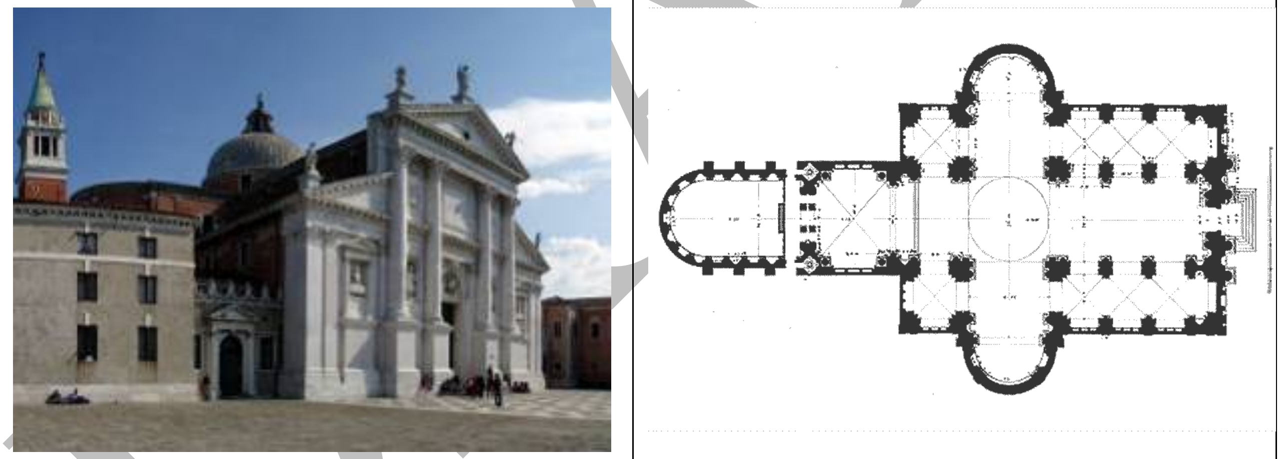


Figure 4.39. Class B1 - Sant'Andrea al Quirinale church in Rome (Italy).

DRAFT

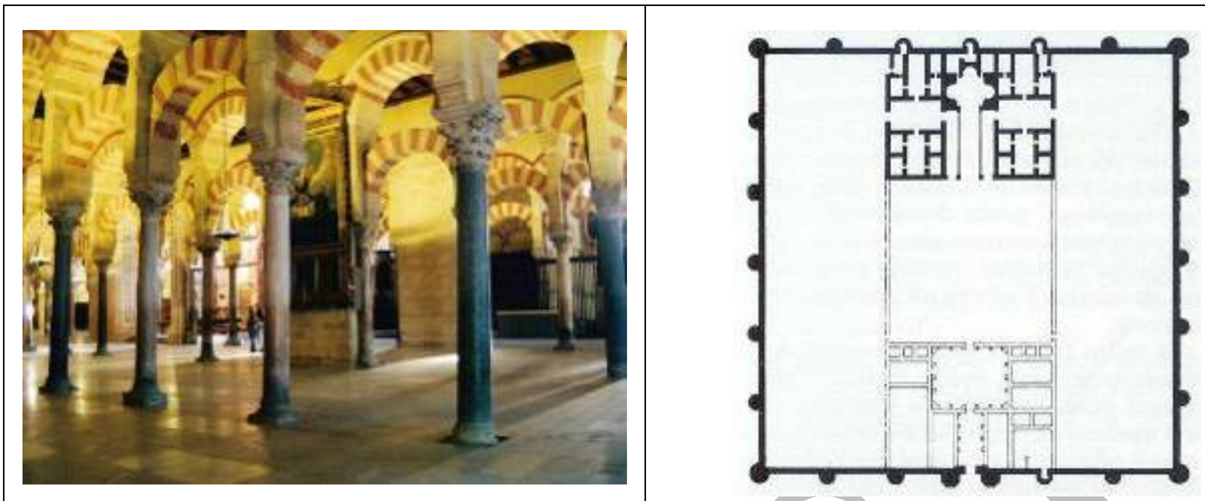


Figure 4.40. Class B2 - Mosque of Cordova (Spain).

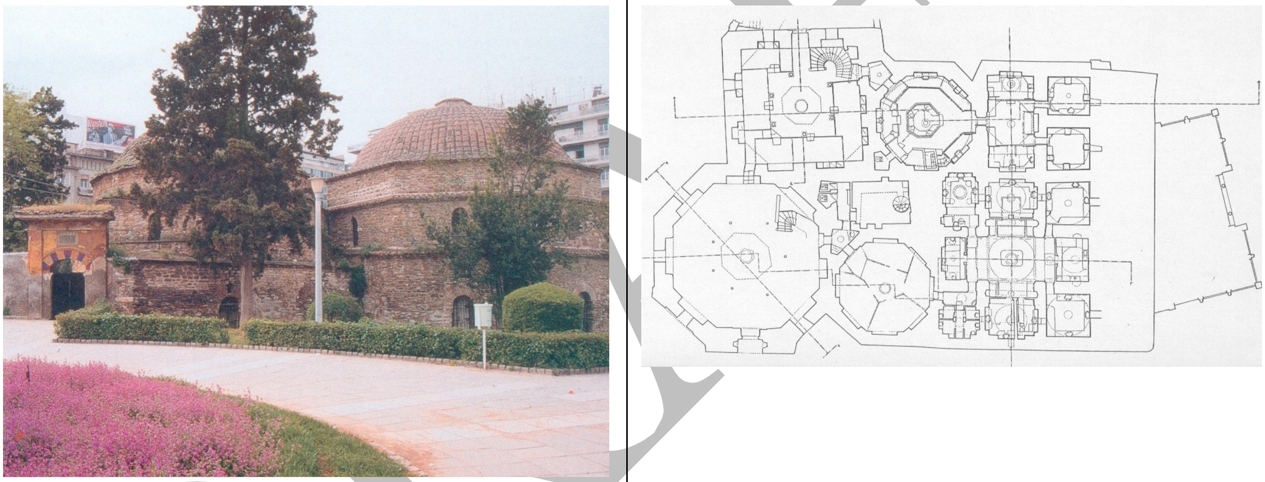
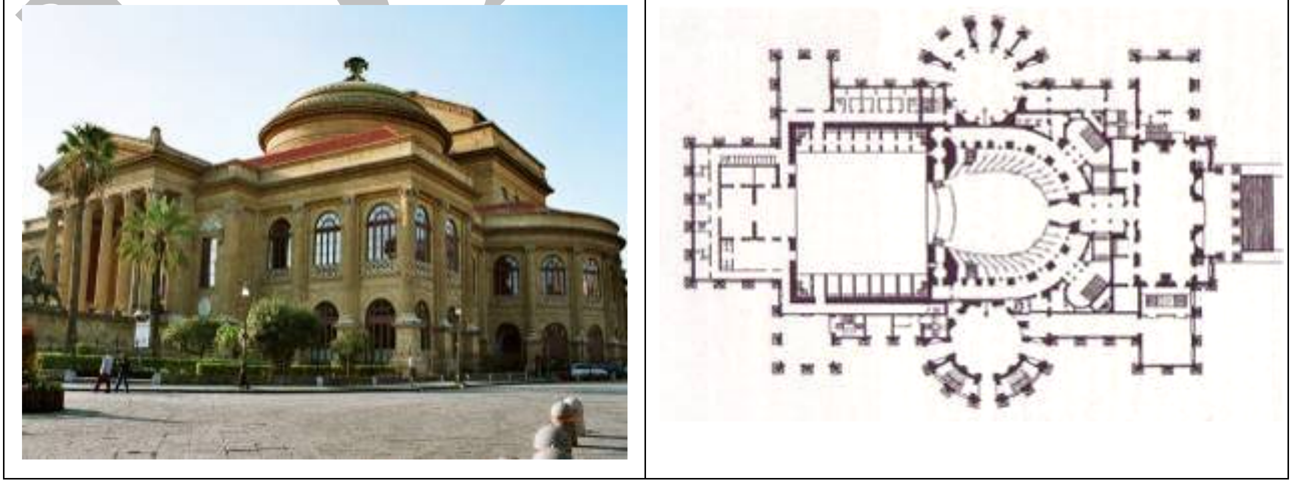


Figure 4.41. Class B5 – Hammam in Thessaloniki (Greece).



DRAFT

Figure 4.42. Class B6 – Massimo Theatre in Palermo (Italy).

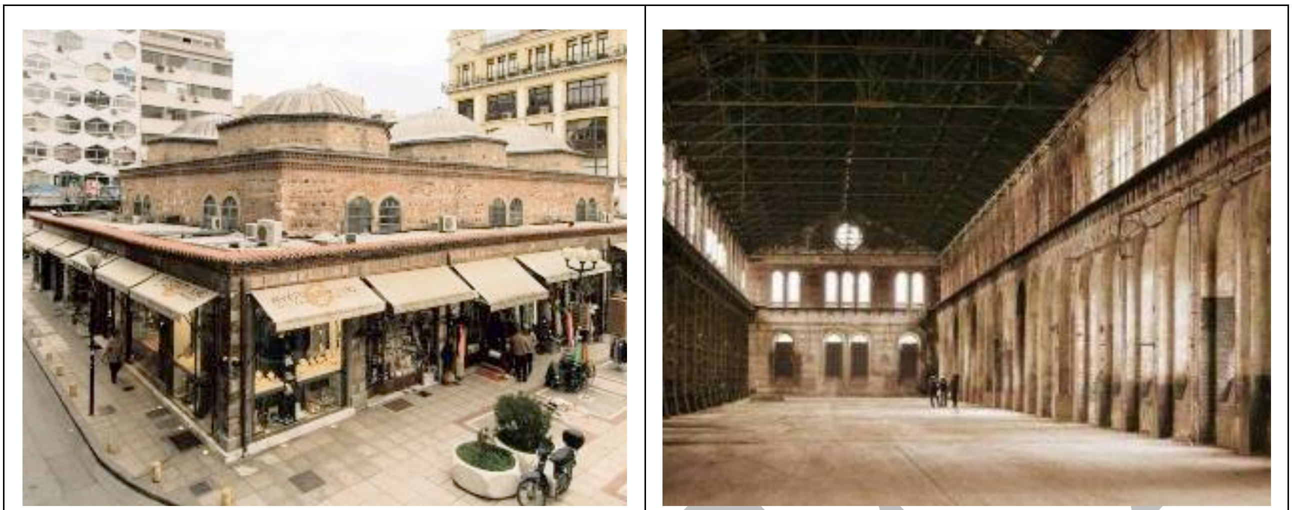


Figure 4.43. Classes B7 and B8 – On the left, bezesteni in Thessaloniki (Greece); on the right, Grandi Officine Ferroviarie in Turin (Italy).

DRAFT

Buildings belonging to class C are slender structures that suffer large bending effects owing to their large masses and high mass centres. Due to their height, such structures are typically subjected to high vertical compressive stresses even under gravity loads; bending derived from seismic actions produce further increases leading the structure to an overall failure in compression. The following types of building present this type of behaviour:

- Towers (C1), intended as military (e.g. defence towers) or civil structures (e.g. clock towers) (Figure 4.44). They usually are squared structures regular in their height, with limited and small openings. Floors are presents at different levels.
- Bell towers (C2), minarets (C3) and lighthouses (C4) (Figure 4.45). These types of slender structures may be of various shape (square, circular or polygonal) but are all characterized by a set of openings concentrated on their top. This is due to the particular functions carried out in their inner spaces. In the case of bell towers and minarets, the openings are motivated by the need to let a sound to be hear from surrounding people (the bell sound in Christian tradition, the call to prayer in Muslim tradition), whereas in lighthouses the openings are useful to let the internal light to be seen from surrounding seamen. From the seismic point of view, the presence of these openings represent a vulnerability factor, since often damage in concentrated in this part of the structure. The need to reach the top of the structure determine the presence of intermediate floors.
- Chimneys (C5). These are typically circular slender structures used to lead the industrial smokes or steams above civil building level (Figure 4.46). The distinctive feature of these buildings is the absence of intermediate floors. This determine a higher slenderness of this type of structures.

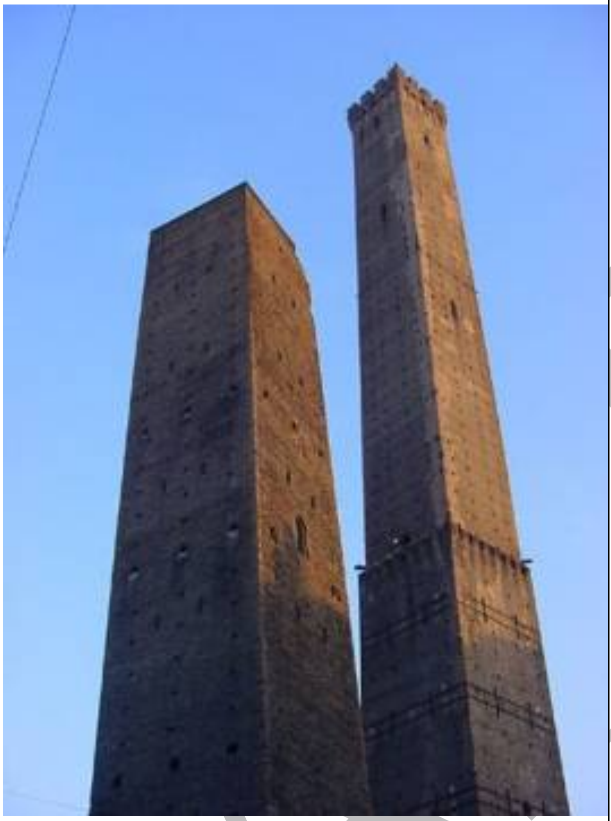


Figure 4.44. Class C1 – Asinelli Towers in Bologna (Italy).

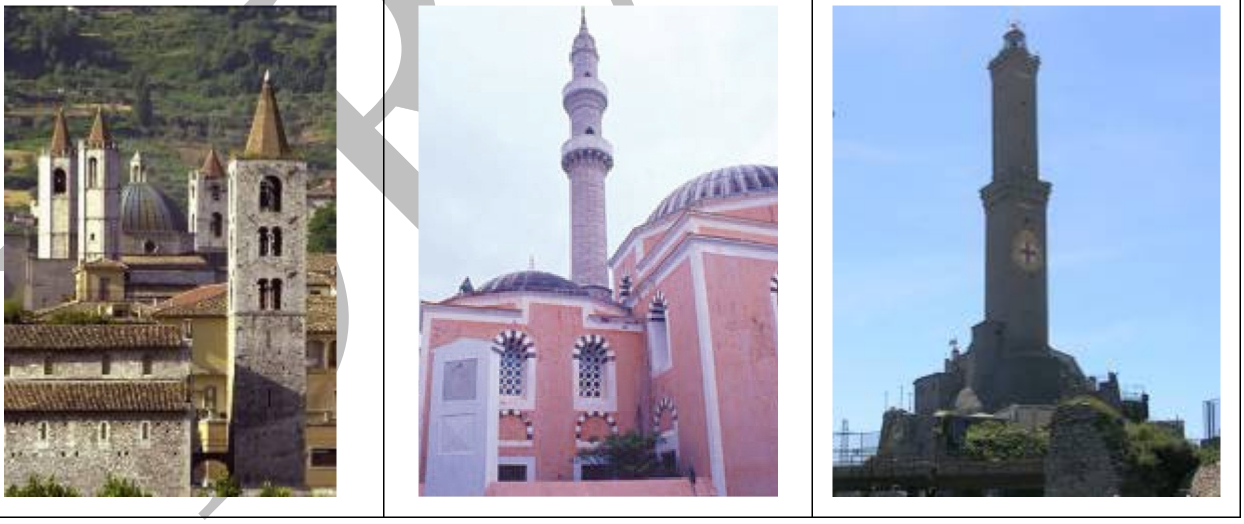


Figure 4.45. Classes C2-C4 – From the left: bell towers in Ascoli Piceno (Italy), minaret of the Suleiman Mosque in Rhodes (Greece) and Lanterna of Genoa (Italy).

DRAFT

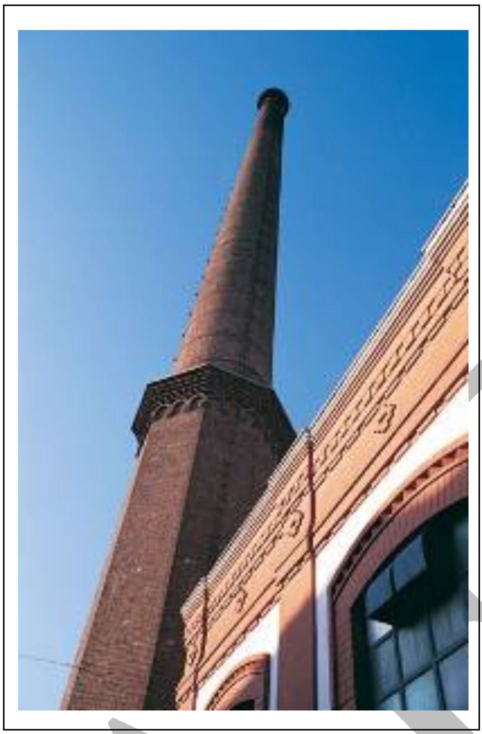


Figure 4.46. Class C5 – Main chimney in Crespi d'Adda factory in Milan (Italy).

All those structures in which arches are the prevalent structural elements are considered in class D. On the one side, reference is made to triumphal arches (D1), aqueducts and bridges (D2 and D3), which are heavy and stiff which do not usually suffer seismic damage (Figure 4.47). On the other side, reference is made to cloisters (Figure 4.48), intended as arched passageways (usually closed on one side) forming a court. These latter structures are more vulnerable from the seismic point of view, due to the presence of light arches and low gravity loads (derived from their limited height).

DRAFT

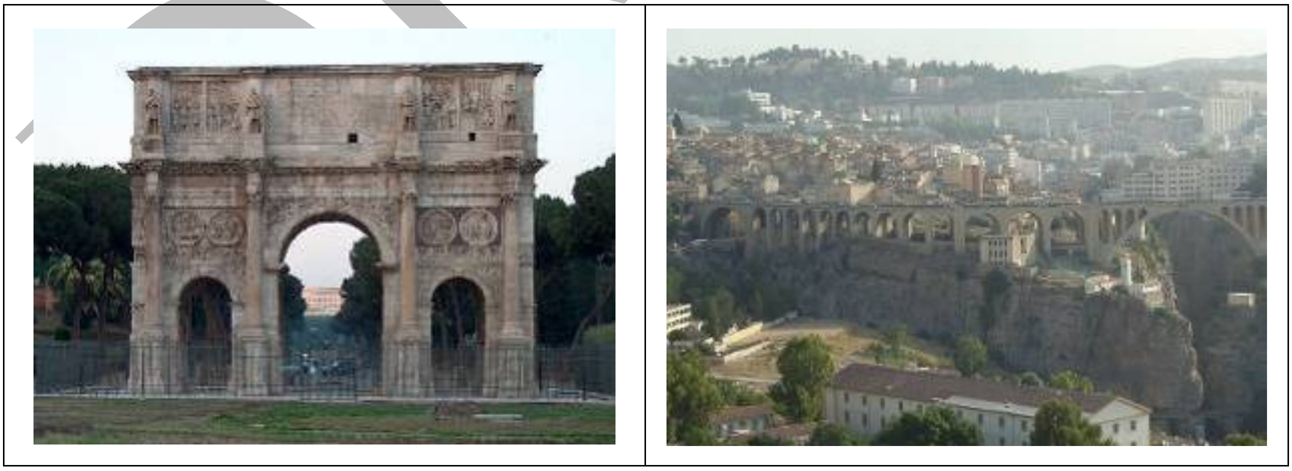


Figure 4.47. Classes D1 and D3 – On the left, Roman triumphal arch in Rome (Italy); on the right, Sidi Rached bridge in Costantine (Algeria).

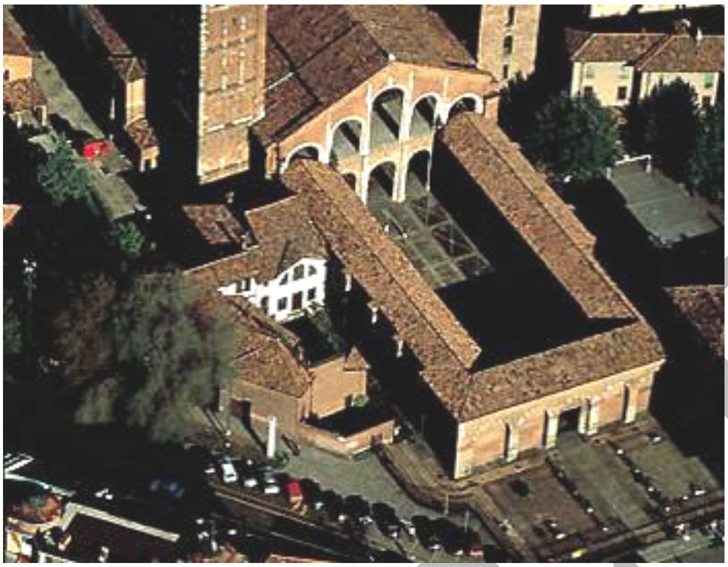


Figure 4.48. Class D4 – Cloisters of S. Ambrogio church in Milan (Italy).

Structures belonging to class E are made of very massive walls which, owing to their thickness, have a three-dimensional behaviour completely different from that of standard walls. Typically, these type of structures suffer local damage such as cracks or spalling due to hydrostatic thrust, detachment of external masonry leaf, partial collapses. Typically, building belonging to this class are fortresses (E1) and defensive city walls (E2) (Figure 4.49). However, also Roman and Greek theatres have been considered in this class (Figure 4.50), since they are very complex and massive masonry structures mainly subjected to local failures.

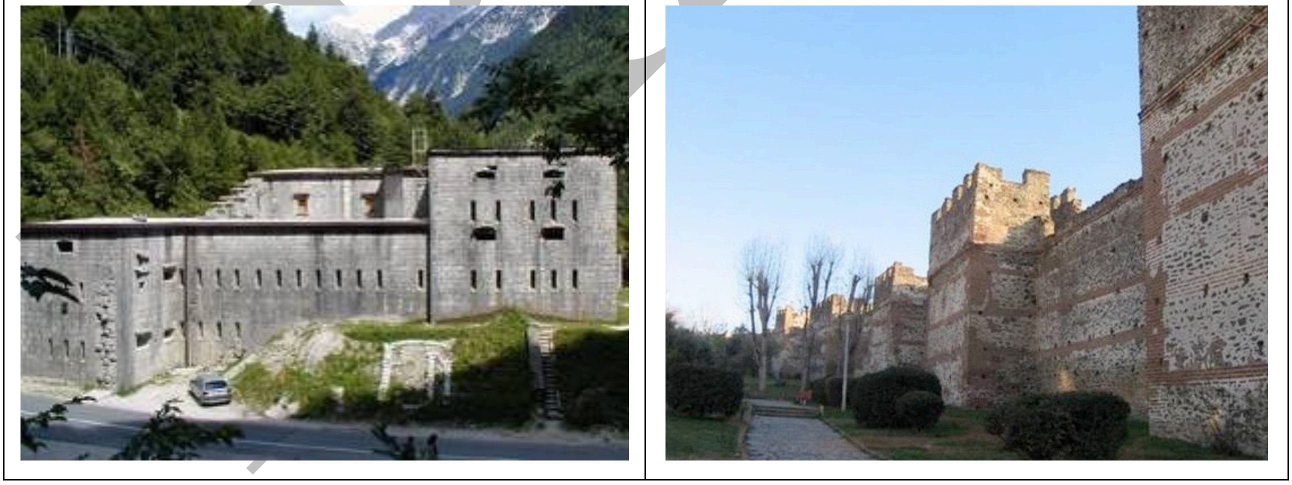


Figure 4.49. Classes E1/E2 – On the left, Kluze Fortress (Slovenia); on the right, defensive city walls of Thessaloniki (Greece).

DRAFT

DRAFT

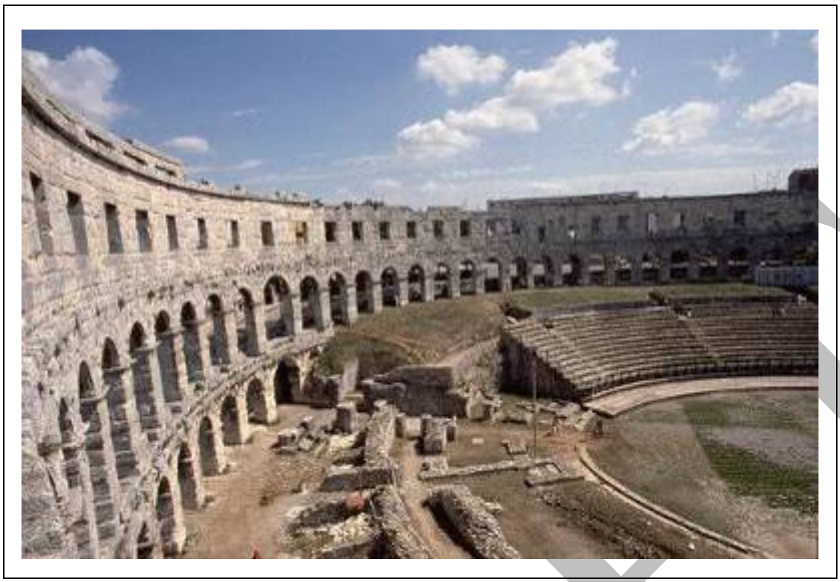


Figure 4.50. Class E3 – Pola amphitheatres (Croatia).

Masonry structures made of very large blocks (in comparison with the size of the structure) are usually subjected to overturning and loss of stability under seismic actions. For this reason, columns (F1), obelisks (F2) and thrilithes (F3) (Figure 4.51), and Greek temples (F4) (Figure 4.52, on the right) are included in the class F. Archaeological ruins (F5) are also considered in this class, referring to those standing out elements that may be subjected to overturning due to their slenderness.

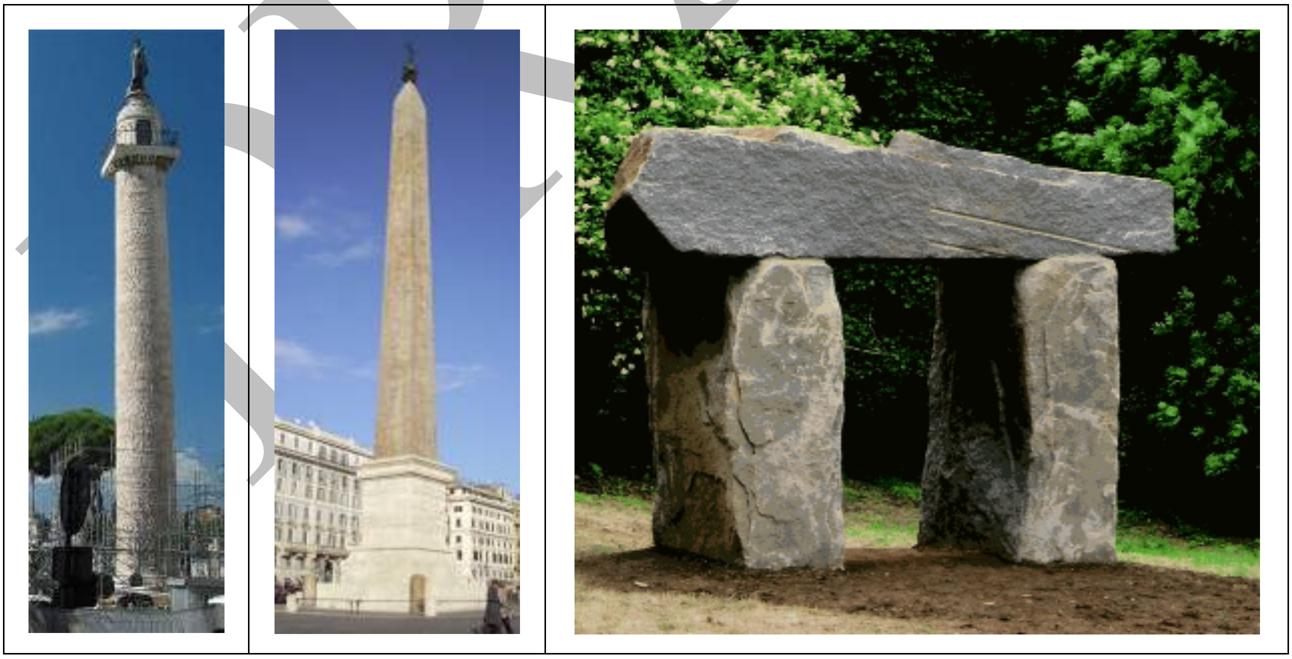


Figure 4.51. Classes F1/F3 – From the left: Colonna Traiana in Rome (Italy), Laterenense Obelisk in Rome (Italy) and Trilithe de kerguehennec (France).

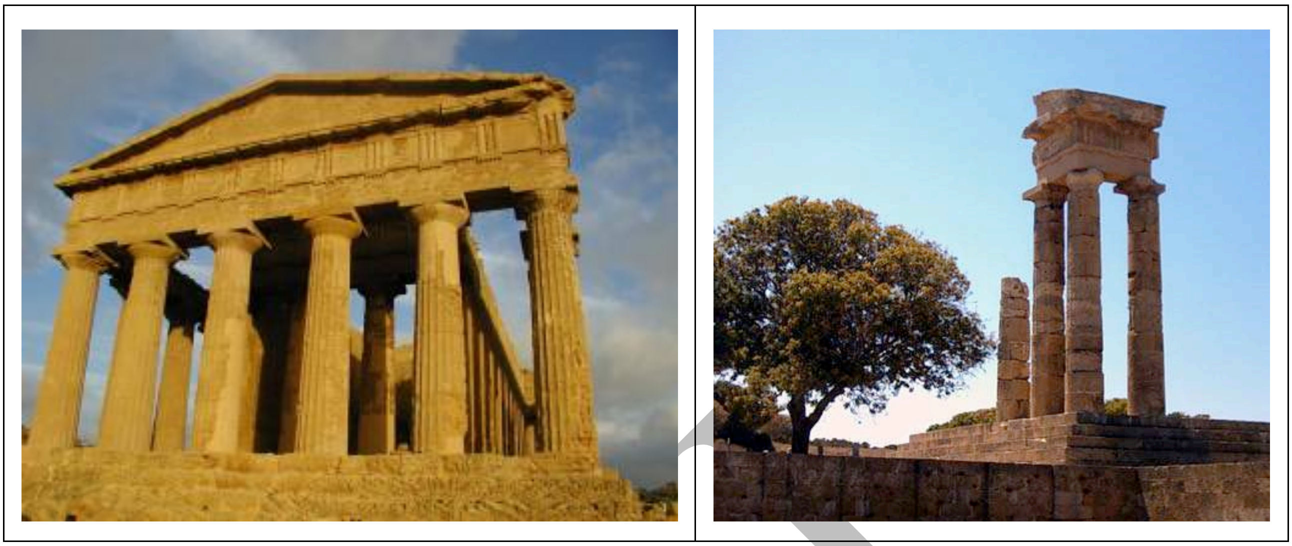


Figure 4.52. Classes F4/F5 – On the left, Greek Temple in Agrigento (Italy); on the right, archaeological site of Monte-Smith close to Rhodes (Greece).

Finally, class G have been defined in order to take into account all those complex systems of building that characterize the historic centre of European and Mediterranean cities. This class refers to ordinary buildings' aggregates, which assume the relevance of cultural heritage asset as whole in the urban context (Figure 4.53). The seismic response and damage modes are similar to that of class A, but interaction among adjacent buildings should be considered.

DRAFT

Figure 4.53. Class G – Kasbah of Algiers (Algeria).

The Table 4.4 shows the correlation between the classes of architectonic assets identified and the classes of damage.

Table 4.4. Correlation between type of buildings and damage classification.

Damage class Architectonic asset class		A	B	C	D	E	F	G	H	I
A	A1 – palaces	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
	A2 – castles	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
	A3 - religious houses	🟥	🟧	🟧	🟧	🟨		🟧	🟧	🟧
	A4 - caravansaries	🟥	🟧	🟧	🟧	🟨		🟧	🟧	🟨
	A5 - collective buildings	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
B	B1 - churches	🟧	🟥	🟧	🟧	🟨		🟧	🟧	🟧
	B2 - mosques	🟧	🟥	🟧	🟧	🟨		🟨	🟧	🟧
	B3 - baptisteries	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B4 - mausoleums	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B5 – hammam	🟧	🟥	🟨	🟧	🟨			🟧	🟧
	B6 - modern theatres	🟧	🟥	🟨	🟧	🟨		🟧	🟧	🟨
	B7 - markets and bazaars	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B8 - industrial buildings	🟧	🟥		🟨	🟨		🟧		
C	C1 – towers	🟧		🟥		🟨		🟨		
	C2 - bell towers	🟧	🟨	🟥	🟧	🟨		🟨		
	C3 - minarets	🟧	🟨	🟥	🟨	🟨		🟨		
	C4 - lighthouses	🟧	🟨	🟥		🟨		🟨		
	C5 - chimneys	🟧		🟥						
D	D1 - triumphal arches				🟥	🟧			🟨	
	D2 - aqueducts		🟨	🟨	🟥	🟧			🟧	
	D3 – bridges		🟨	🟨	🟥	🟧			🟧	
	D4 - cloisters				🟥	🟨			🟧	
E	E1 - fortresses					🟥		🟨		
	E2 - defensive city walls					🟥				
	E3 - Roman and Greek theatres			🟨	🟧	🟥	🟧			

🟨 Occasional behaviour
🟧 Possible behaviour
🟥 Prevailing behaviour

DRAFT

F	F1 - columns						🟥				
	F2 – trilithes						🟥				
	F3 - obelisks						🟥				
	F4 - archaeological ruins				🟧	🟨	🟥				
	F5 - Greek temples						🟥				
G	Historical centers	🟧	🟧	🟨	🟨	🟨		🟧	🟧		

3.2.11 Macroelements

Damage in masonry buildings is typically localized in a set of main building elements, that have a typical seismic behaviour and that will be here named "macroelements". In Tables 4.5-4.10, macroelements related to each class of architectonic asset are catalogued. In Figures 4.54-4.57, some examples are presented.

Table 4.5. Macroelements in architectonic asset class B.

MACROELEMENT	CLASS B							
	B1	B2	B3	B4	B5	B6	B7	B8
façade	■	■				■	■	■
Hall	■	■	■	■		■	■	■
central nave	■							
lateral nave	■							
transept	■							
triumphal arch	■							
tympanum	■	■		■		■		
apse/presbytery	■		■					
chapel	■		■	■				
prothyrum/narthex	■	■	■			■		
Roof	■	■	■	■	■	■	■	■
Dome	■	■	■	■	■	■	■	
self standing elements	■	■	■		■	■		
jutting out elements	■	■	■			■		
internal colonnade			■	■	■	■		■
external colonnade			■	■		■		

DRAFT

added buildings	■	■		■	■	■	■	■
staircase	■			■		■		
patio/cloister		■						
Loggia						■		

Table 4.6. Macroelements in architectonic asset class F

MACROELEMENT	CLASS F
	F1/F5
horizontal blocks	■
vertical blocks	■

Table 4.7. Macroelements in architectonic asset classes A, E1 and G.

MACROELEMENT	CLASS A	CLASS E	CLASS G
	A1/A5	E1	
Walls (façades, internal walls)	■	■	■
Floors	■	■	■
Vaults	■	■	■
colonnade, loggia, cloister	■		
Staircase	■	■	■
Roof	■	■	■
Dome	■		
jutting out elements	■	■	■
self standing elements	■	■	■
added building	■	■	■

Table 4.8. Macroelements in architectonic asset class C.

MACROELEMENT	CLASS C
	C1/C5
main body	■
upper part	■
self standing elements	■
jutting out elements	■

DRAFT

Table 4.9. Macroelements in architectonic asset class D.

MACROELEMENT	CLASS D	
	D1/D3	D4
Arcade	■	
Vault	■	■
Pier	■	
Spandrel	■	
Internal colonnade		■
Walls/external colonnade		■

Table 4.10. Macroelements in architectonic asset sub-classes E2/E3.

MACROELEMENT	CLASS E
	E2/E3
self standing elements	■
jutting out elements	■
main wall	■

DRAFT

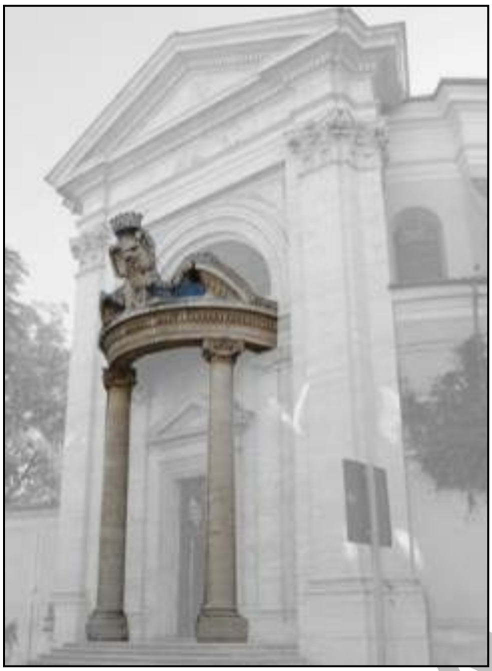
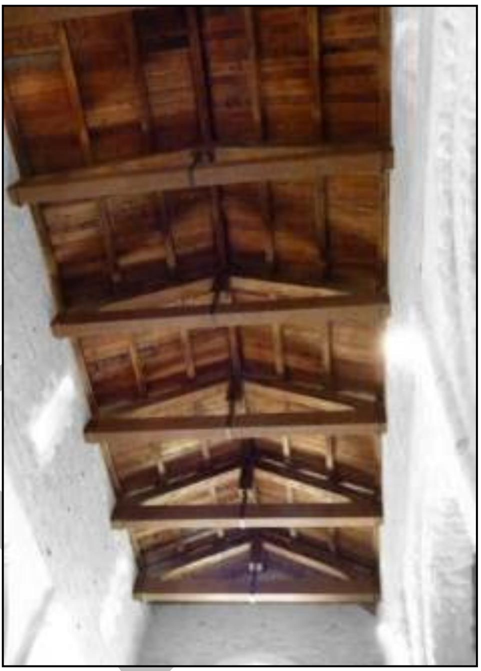
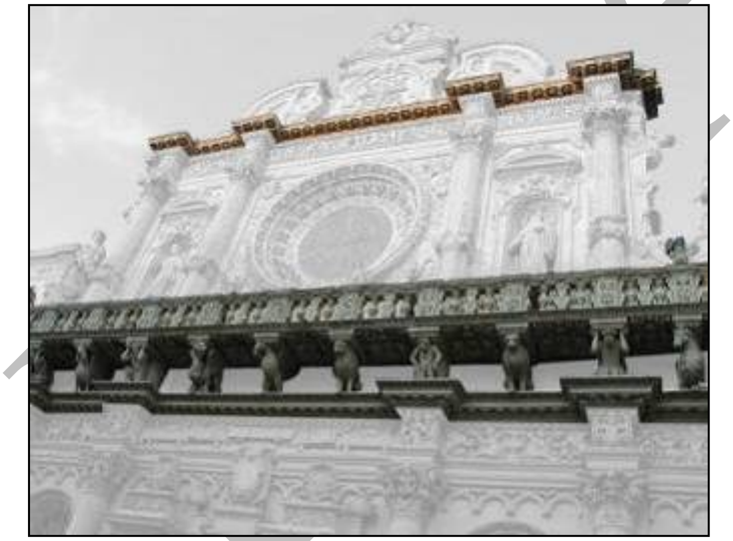
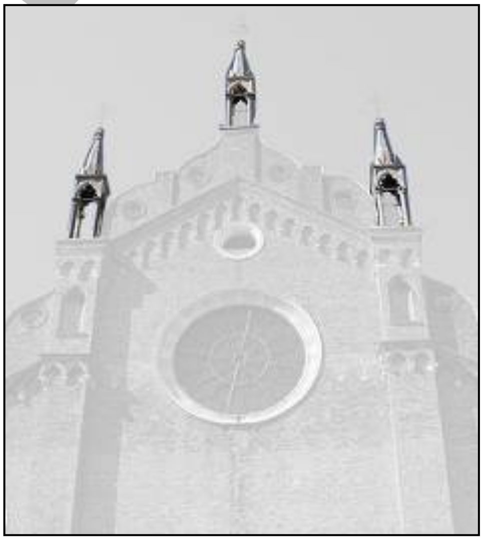
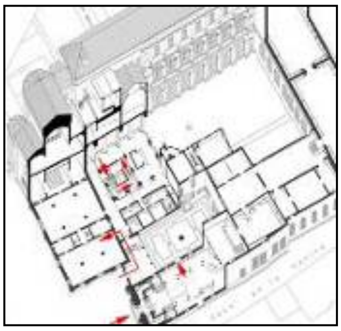
Prothyrum/narthex	Roof
	
Jutting out elements	Self standing elements
	

Figure 4.54. Examples of macroelements for class B1.

Main building	Portico, loggia, cloister
	
Staircase	Jutting out elements
Self standing elements	Dome
Roof	Added buildings

DRAFT

Figure 4.55. Examples of macroelements for class A1/A6, E1 and G.

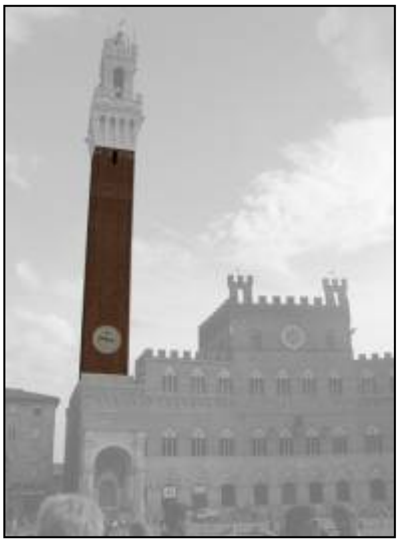
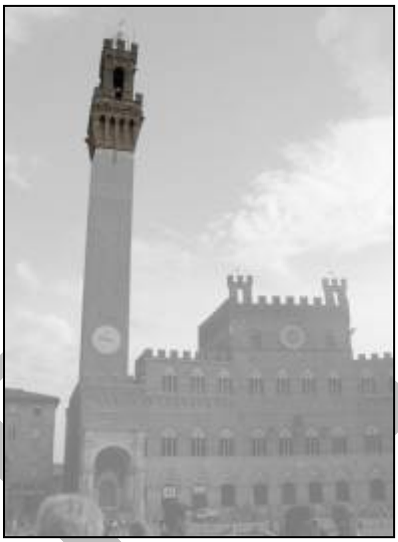
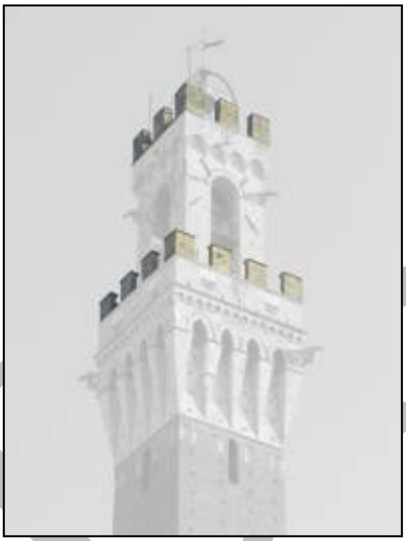
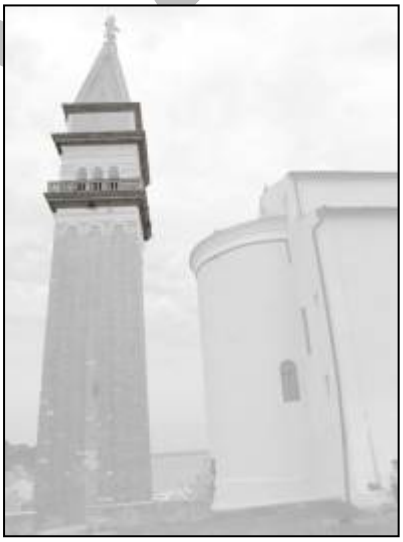
Main body	Upper part
	
Self standing elements	Jutting out elements
	

Figure 4.56. Examples of macroelements for class C.

DRAFT

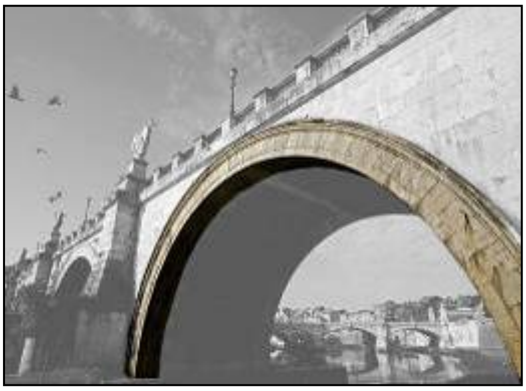
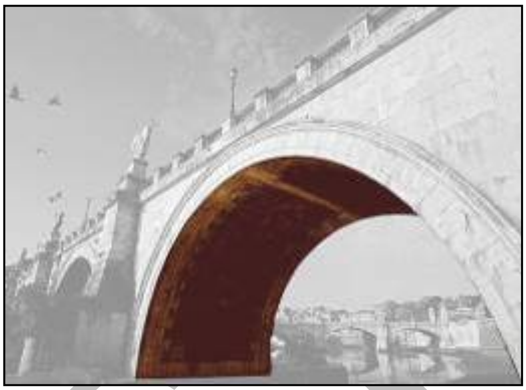
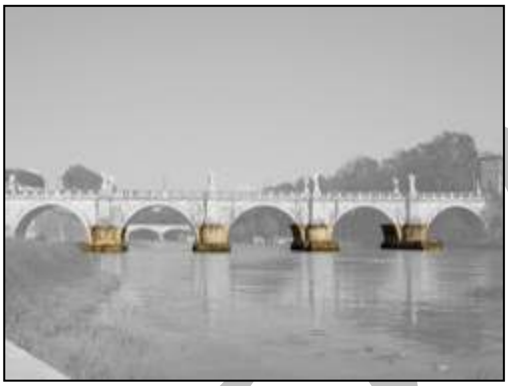
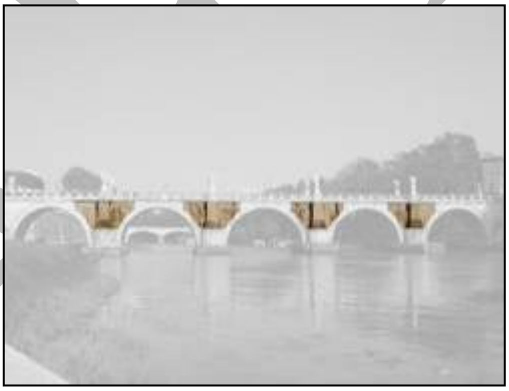
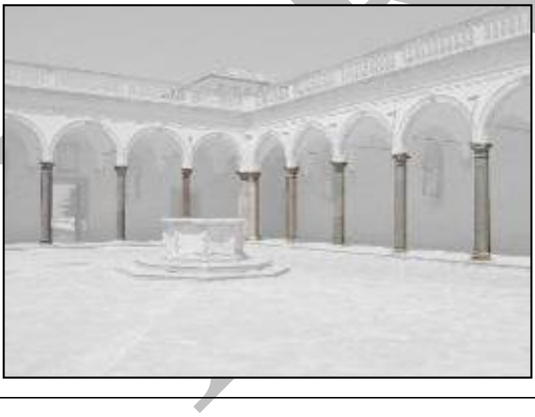
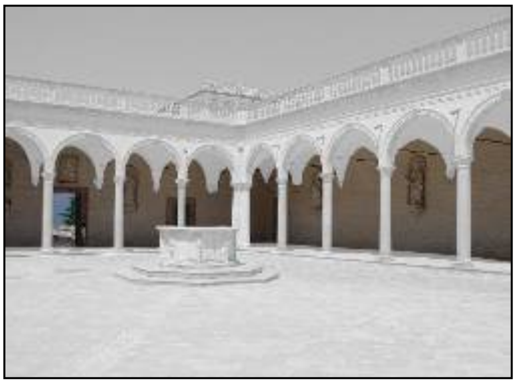
Arcade	Vault
	
Pier	Spandrel
	
Internal colonnade	Walls/External colonnade
	

Figure 4.57. Examples of macroelements for class D.

DRAFT

3.2.12 Correlations among architectonic assets, macroelements and damage

In this paragraph, the correlation between classes of architectonic assets, their macroelements and the most typical damage to them is proposed. Tables 4.12-4.18 are only qualitative and based more on the presence of macroelements than on frequency of damage. Statistical distributions of seismic damage for the different typologies should be very useful.

Table 4.12. Damage classes in macroelements of asset class A

Damage / Macroel.	A				B		C	D	E			F	G			H	I
	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Walls (façades, internal walls)	🟧	🟧	🟧	🟧	🟧	🟧			🟧	🟧	🟧						
Floors													🟧	🟧	🟧	🟧	
Vaults								🟧			🟧					🟧	
Colonnade, cloister, loggia					🟧		🟧	🟧	🟧		🟧		🟧		🟧	🟧	
Staircase	🟧			🟧				🟧					🟧	🟧	🟧	🟧	
Roof													🟧	🟧	🟧		
Dome																	🟧
Jutting out elements												🟧					
Self standing elements	🟧	🟧	🟧	🟧	🟧	🟧		🟧	🟧		🟧	🟧	🟧	🟧	🟧	🟧	
Added building	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧		🟧	🟧	🟧	🟧	

DRAFT

Table 4.13. Damage classes in macroelements of asset class B

Damage / Macroel.	A				B		C	D	E			F	G			H	I
	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Facade	🟧			🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧						
Tympanum	🟧			🟧	🟧	🟧			🟧		🟧						
Hall	🟧			🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧					🟧	
Central nave	🟧			🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧					🟧	
Triumphal arch							🟧	🟧									
Apse/ presbitery	🟧			🟧	🟧	🟧			🟧	🟧	🟧		🟧	🟧	🟧	🟧	🟧
Transept	🟧			🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧					🟧	
Lateral nave	🟧			🟧	🟧	🟧		🟧	🟧	🟧	🟧					🟧	
Prothyrum/ narthex				🟧			🟧	🟧	🟧								
Roof													🟧	🟧	🟧		
Chapel	🟧								🟧	🟧	🟧						🟧
Self standing elements	🟧							🟧				🟧					
Dome	🟧			🟧	🟧			🟧	🟧	🟧	🟧		🟧	🟧	🟧	🟧	
Jutting out elements	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	

DRAFT

DRAFT

Table 4.14. Damage classes in macroelements of asset class C

Damage / Macroel.	A				B		C	D	E			F	G			H	I
	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Main body	🟧	🟧	🟧	🟧			🟧		🟧	🟧			🟧	🟧	🟧		
Upper part	🟧			🟧				🟧	🟧	🟧	🟧		🟧	🟧	🟧	🟧	🟧
Self standing element	🟧	🟧	🟧	🟧	🟧	🟧		🟧	🟧		🟧	🟧	🟧	🟧	🟧	🟧	
Jutting out element	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧		🟧	🟧	🟧	🟧	

Table 4.14. Damage classes in macroelements of asset sub-classes D1/D3

Damage / Macroel.	A				B		C	D	E			F	G			H	I
	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Arcade								🟧	🟧								
Vault								🟧								🟧	
Pier							🟧			🟧							
Spandrel										🟧							

Table 4.15. Damage classes in macroelements of asset sub-classes D4

Damage / Macroel.	A				B		C	D	E			F	G			H	I
	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Vault								🟧								🟧	
Internal colonnade							🟧	🟧									

Walls or external colonnade	🟧	🟧	🟧	🟧			🟧	🟧	🟧	🟧	🟧		🟧	🟧	🟧		

Table 4.16. Damage classes in macroelements of asset sub-classes E2/E3

Damage	A				B		C	D	E			F	G			H	I
Macroel.	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Main wall									🟧								
Jutting out elements												🟧					
Self standing elements	🟧			🟧				🟧	🟧		🟧	🟧	🟧	🟧	🟧	🟧	

Table 4.17. Damage classes in macroelements of asset class F

Damage	A				B		C	D	E			F	G			H	I
Macroel.	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Single main building elements								🟧	🟧			🟧					

Table 4.18. Damage classes in macroelements of asset classes E1 and G

Damage	A				B		C	D	E			F	G			H	I
Macroel.	A-a	A-b	A-c	A-d	B-a	B-b			E-a	E-b	E-c		G-a	G-b	G-c		
Walls (façades, internal walls)	🟧	🟧	🟧	🟧	🟧	🟧			🟧	🟧	🟧						
Floors													🟧	🟧	🟧	🟧	
Vaults								🟧			🟧					🟧	
Colonnade cloister loggia					🟧		🟧	🟧	🟧		🟧		🟧		🟧	🟧	
Staircase	🟧			🟧				🟧					🟧	🟧	🟧	🟧	
Roof													🟧	🟧	🟧		

DRAFT

Jutting out elements												🟧					
Self standing elements												🟧					
Added building	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧	🟧		🟧	🟧	🟧	🟧	

3.3 *Review of models for masonry constructions*

3.3.1 *Classes of models*

Models developed in literature for masonry structures will be here classified following two criteria (Table 4.19): scale of analysis (whether material or structural element one) and type of description of masonry continuum (whether continuous or discrete). Figure 4.58 shows some examples. It is well evident that this classification is not exhaustive, since not all the models presented in the literature can be included in the considered classes and hybrid cases are always possible.

Table 4.19. Types of models for masonry structures.

Type Scale	Continuous models	Discrete models
Material scale	Continuum constitutive laws	Interfaces models
Element scale	Structural elements models	Macro-blocks models

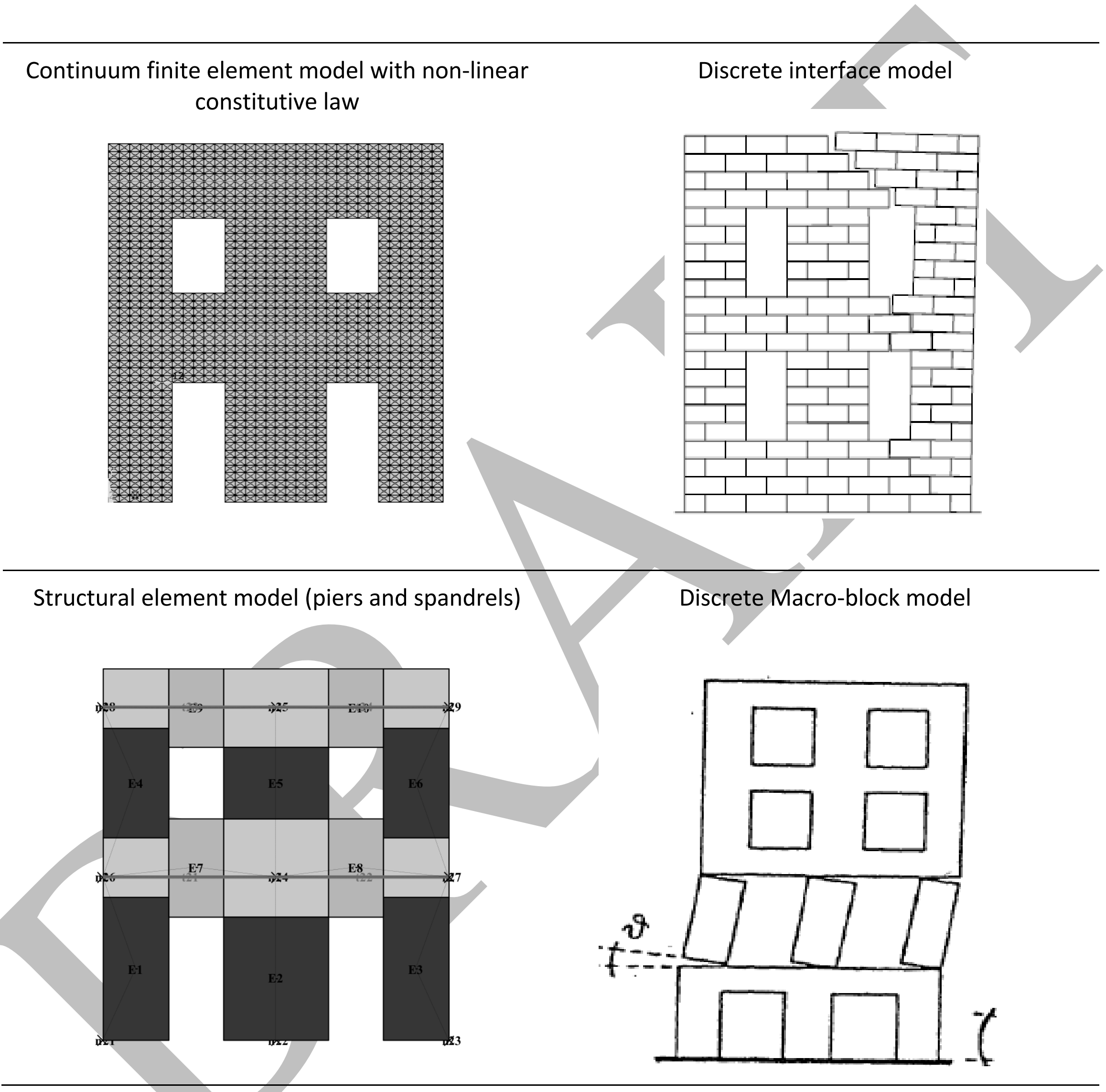
Models developed at *material scale* are oriented to describe in an accurate way the complex behaviour of masonry solids. At this scale, a fundamental role is played by the composite nature of the material, which may be considered whether like heterogeneous or homogenous.

If it is considered as heterogeneous, *discrete interface models* are usually adopted. This means that each single constituent of the material (blocks and joints) is modelled separately and then assembled with the others by mean of interface elements. This is a very accurate modelling approach, but may require large computational efforts.

If the material is considered as homogeneous, a more synthetic description of the material is provided, by mean of *continuum constitutive laws*. To define these laws, two approaches are usually adopted. In the phenomenological approach, the behaviour of the material is defined on the basis of experimental tests, which directly provide stress-strain relationships and limit domain. In the micromechanical approach, the global mechanical response of the composite material is usually obtained adopting homogenization procedures, that is studying a representative volume element (RVE) of the heterogeneous material and then determining the constitutive laws of the homogenized equivalent material. Despite being less detailed, such models may be extended to larger structural portions and represent a valid option also for complex geometries.

DRAFT

Non-linear modelling and analysis of entire buildings require, however, a further level of synthesis due to the high computational cost intrinsic in continuous non-linear laws. The driving idea of models developed at *element scale* is to identify, within the masonry continuum, portions of structure subjected to recurrent damage modes (to this aim, post-earthquake damage observation is a remarkable source of information). The masonry structure is thus not seen a "blurred" continuum but as a set of bodies with common mechanical behaviours. Also at this scale of analysis, two different approaches are usually adopted.


Continuum finite element model with non-linear constitutive law

Discrete interface model

Structural element model (piers and spandrels)

Discrete Macro-block model

Figure 4.58. Examples of different modelling approaches to simple walls.

The first one is a *discrete* approach, in which the behaviour of a discrete set of masonry bodies, connected through interfaces, is considered. The shape of each body is defined on the basis of recurrent crack patterns observed in post-earthquake surveys. Each masonry body is commonly assumed as rigid or elastic (only in some cases, the compression domain is limited by an elastic-plastic of rigid-plastic law). Non-linear behaviour is concentrated in interfaces, which are assumed as non-resisting tension joints, capable, in some cases, to transfer frictional forces.

DRAFT

The second one is a *continuous* approach requiring the identification of macroscopic structural elements. Such elements are defined from the geometrical and kinematic point of view through finite elements such as shells or frames, and correspond to masonry portions such as masonry "piers", "spandrels", panels or architectural elements with specific boundary conditions. The static equilibrium of the element can be formulated with reference to the internal force resultants instead of the continuum stress. The macro-elements represent damage, cracking, sliding and rotations in predefined zones which are characterised based on mechanical assumptions and implementation of, more or less sophisticated, non-linear constitutive laws. It is worth noting that these models are oriented to evaluate the overall response of masonry constructions by considering the response of structural elements to only in-plane forces.

The use of macro-models for the nonlinear analysis of masonry structures has been introduced in several guidelines. The macro-models proposed in the literature have the advantage to be characterized by few parameters and reduced computational efforts regarding the modelling and the structural analysis phases. Nevertheless, these models assume that the damage of elements can occur only in the concentrated zones established by the user before the analysis, while the other parts of the elements remain undamaged. This aspect could be inaccurate for the case of strengthened structures which may be characterized by damaged configurations quite different from the case of the unstrengthened one.

3.3.2 *Continuum constitutive laws*

Continuum constitutive laws are commonly adopted in conjunction with two or three-dimensional Finite Element Models, which allow to perform both incremental and collapse analyses. As it has been previously stated, whether a *phenomenological* or *micromechanical* approach is commonly followed to define these models.

In the context of *phenomenological* approach, *smeared cracking* and *plasticity laws* have been widely used in past (in particular to model non-periodic and irregular masonries, for which homogenization techniques result particularly difficult). The *smeared crack* approach was originally developed to model the fracture behaviour of reinforced concrete structures that may exhibit diffuse crack pattern (see for a review De Borst et al. 1993 and 1994). Specific applications to masonry, taking into account the anisotropic behaviour of this material, have been made by Ali and Page (1988), Lofti and Shing (1991) and Rots (1991). In the field of *plasticity laws*, NRT ones played a relevant role in past. However, despite several studies made to solve the theoretical problem of the equilibrium of bidimensional traction non-resistant solids subjected to external forces (Romano and Romano 1979, Como e Grimaldi 1985, Giaquinta e Giusti 1985, Panzeca and Polizzotto 1988, Del Piero 1989, Di Pasquale 1992), only few applications to incremental analyses were made (Romano e Sacco 1984, Sacco 1990, Di Pasquale 1992, Cuomo and Ventura 2000). More complex anisotropic plasticity laws were developed in recent past on the basis of direct experimentation (Lourenço et al. 1997 and 1998, Lourenço 2000) or basic mechanical observations (Papa and Nappi 1997, Berto et al. 2002). Another interesting model based on the *critical plane* approach has been developed by Pietruszczak and Ushaksaraei (2003). This latter approach consists of finding, at each step of the incremental analysis, the plane for which a failure function reaches a maximum and of determining the inelastic deformations on such plane. The main drawback of these type of models is calibration of mechanical parameters, that should be obtained from masonry tests of sufficiently large size under homogeneous states of stress.

DRAFT

In the *micromechanical* approach, the global mechanical response of the composite material is usually obtained adopting *homogenization procedures* or *multi-scale analyses*.

Based on the study of a Representative Volume Element (RVE) or Unit Cell (UC) of the heterogeneous material, *homogenization procedures* aim at determining constitutive laws of the homogenized equivalent material. In the last decades, many contributes have been presented in this field for masonry, with reference to both elastic and collapse range.

In the elastic range, relevant results are reported in (Pande et al. 1989, Anthoine 1995, Lourenço and Rots 1997, De Felice 2001, Cecchi and Sab 2002a) and (Cecchi and Sab 2002b, Cecchi et al. 2005) respectively for in- and out-of-plane actions. In these models, masonry is considered as a regular periodic material and blocks and joints are assumed as linear elastic materials. A simplified model, which allows to obtain in explicit form masonry constitutive equations, is based on the hypothesis of joints reduced to interfaces with zero thickness. A further simplification, which allows to determine easily the contribution of head and bed joints on the overall mechanical properties of masonry, consists in the assumption of rigid blocks (Cecchi and Sab 2002a, Cecchi and Sab 2002b). Such hypothesis results particularly true from a practical point of view, since joints are more deformable than blocks. As far as irregular non-periodic masonry is concerned, there is still a lack of literature. In this framework, a recent work by Cluni e Gusella (2004) results particularly interesting, since it has been shown that irregular textures can be treated at a micro-scale by means of an elementary cell with variable dimensions. Starting from the studies on homogenization procedures for non-periodic bodies (Papanicolaou and Varadhan 1979, Kozlov 1980, Sab 1992, Lachihab and Sab 2005), Cecchi e Sab (2008) proposed a homogenization technique for masonry with regular non periodic texture, by means of a perturbative approach under 2D plane and linear assumptions.

In the collapse phase, references (Spence and Coburn 1992, Alpa and Monetto 1994, Corigliano and Maier 1995, De Buhan and De Felice 1997, Ferris and Tin-Loi 2001, Sutcliffe et al. 2001, Milani et al. 2006, Cecchi et al. 2007, Cecchi and Milani 2008) result particularly important for the determination of masonry failure domain of masonry regular periodic texture, in case of both in- and out-of-plane actions. The basic assumption of many of these models is blocks infinite strength and the cohesive-frictional behaviour of joints (Mohr-Coulomb failure criterion).

All these techniques present the advantage of keeping memory, at material scale, of the main characteristics of masonry micromechanics. Despite some attempts have been made to extend homogenization procedures to non-linear field (Maier et al. 1991, Pietruszczak and Niu 1991, Lee et al. 1996, Pegon and Anthoine 1997), the complexity of this technique does not allow, today, the formulation of rigorous evolutive non-linear constitutive laws to be adopted in incremental analyses.

An interesting approach for solving the nonlinear homogenization problem is the so-called Transformation Field Analysis (TFA), proposed initially by Dvorak (1992) and applied to plasticity and visco-plasticity problems by Fish and Shek (2000). The use of the TFA requires the computation of localization and transformation tensors. When the geometry of the microstructure is complex, as in the case of masonry material, numerical techniques can be adopted. In fact, the finite element method or the fast Fourier transform technique are able to accurately evaluate local stress and strain fields, so that the correct non-linear behaviour of the phases can be described. Recently, Sacco (2009) presented a nonlinear Cauchy homogenization procedure for masonry based on TFA, making use of the superposition of the effects and of the finite element method.

DRAFT

The *multi-scale* technique consists in the structural modelling through two different scales: one scale at the continuum mechanics structural level and one scale at the material level able to distinguish the single heterogeneities that are present in the masonry material. The development and the use of multi-scale procedures is a complex task as it is necessary to solve the micromechanical problem and to adopt the obtained results in order to perform the structural analysis. Several *multi-scale* procedures and techniques have been proposed in literature for different materials and structures (Fish and Wagiman 1993, Hughes 1995, Feyel and Chaboche 2000, Nadler et al. 2006). A numerical technique able to account for the possible kinematical coupling between the different scales has been recently presented for the analysis of masonry structures by Brasile et al. (2007). More simplified *multi-scale* techniques, in which the microstructural behaviour of masonry is related to the continuum through simplified micromechanical analysis have been presented by Gambarotta and Lagomarsino (1997), Luciano and Sacco (1997), Massart et al. (2004), Calderini and Lagomarsino (2006), Mistler et al. (2007), Calderini and Lagomarsino (2008) (Figure 4.59).

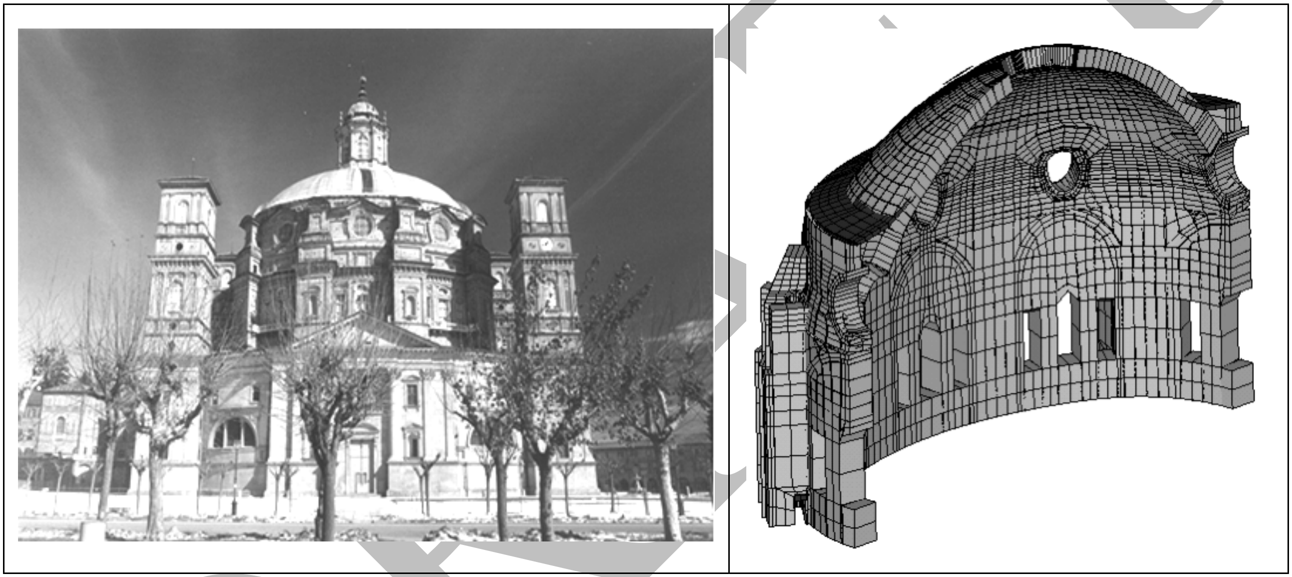


Figure 4.59. The finite element model of Vicoforte's dome by Calderini and Lagomarsino (2008).

All the above cited constitutive models have been developed by describing masonry as a Cauchy continuum, for which two main drawbacks may be pointed out: it does not allow to keep into account the absolute size of the microstructure, and to describe scale effects; the macroscopic fields of the RVE are supposed non-uniform. In order to overcome such drawbacks, various authors have proposed models based on generalized continua. Particular attention has been paid to the Cosserat continuum, in which an internal scale parameter is considered (Masiani et al. 1995, Cerrolaza et al. 1999,Trovalusci and Masiani 2003, Casolo 2006, Brasile et al. 2007).

3.3.3 Structural elements models

This approach aims at evaluating the overall response of masonry structures made up of walls with regular openings, describing with adequate accuracy the in-plane behaviour of single structural elements. The technique is based on the identification of macroscopic structural elements (portions of structure such as

DRAFT

"piers" or "spandrels"), defined from a geometrical and cinematic point of view through finite elements (shell or frame) and described from a static point of view through their internal forces. Piers are the principal vertical resistant elements for both dead and seismic loads; spandrels, which are intended to be those parts of walls between two vertically-aligned openings, are secondary horizontal elements, coupling piers in the case of seismic loads. By concentrating damage, slidings and rotations in predefined structural elements, these models enable one to perform non-linear incremental collapse analyses of entire buildings.

In the field of structural element models, "equivalent frame" models are the most widely diffused. They consider the walls as an idealized frame, in which deformable elements (piers alone or piers and spandrels) connect rigid nodes (parts of the wall which are not usually subjected to damage). Figure 4.60 illustrates a sketch aimed to represent the wall idealisation into an assemblage of structural elements; in particular, different solutions are illustrated according to more simplified models (such as "strong spandrels-weak piers" and "weak spandrels-strong piers" models proposed in FEMA 356 or FEMA 306), for which the actual modelling of spandrels results un-requested, or models (such as "equivalent frame") which consider both pier and spandrel elements.

The modelling of the whole structure is obtained by assembling masonry walls (idealized as 2D frames) and horizontal floors (that may be assumed as rigid or deformable).

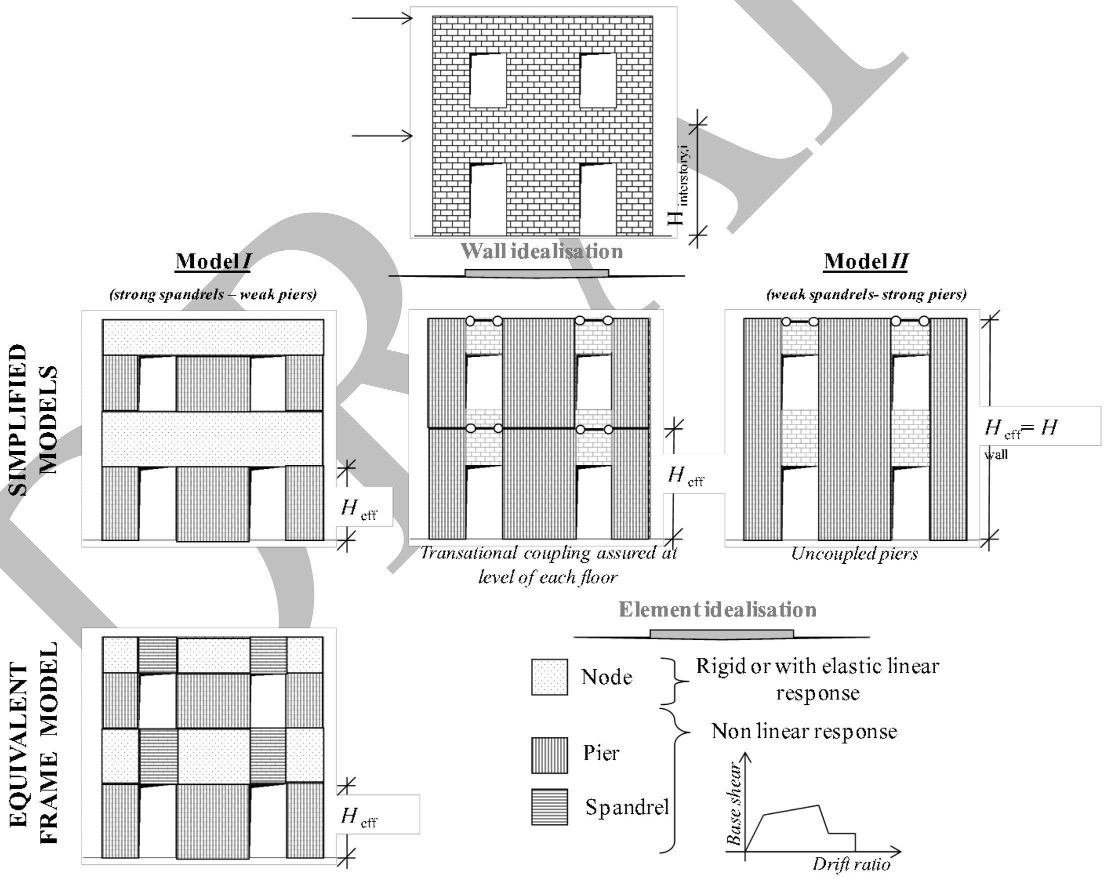

Wall idealisation
$H_{interstory,i}$
Model I
(strong spandrels – weak piers)
Model II
(weak spandrels- strong piers)
SIMPLIFIED MODELS
H_{eff}
H_{eff}
$H_{eff} = H_{wall}$
Transational coupling assured at level of each floor
Uncoupled piers
Element idealisation
EQUIVALENT FRAME MODEL
H_{eff}
Node
Pier
Spandrel
Rigid or with elastic linear response
Non linear response
Base shear
Drift ratio

Figure 4.60. Examples of different modelling approaches to simple walls (from Cattari and Lagomarsino 2008).

DRAFT

Once having idealised the masonry wall into an assemblage of structural elements, the reliable prediction of its overall behaviour mainly depends on the proper interpretation of the single panel. Masonry elements in which the non-linear response is concentrated may be described as *monodimensional* or *bidimensional* structural elements, by mean of more or less detailed models.

In the class of *monodimensional* models, three main different approaches to the modelling of walls have been proposed: the *equivalent strut* idealization, the *non-linear beam* idealization and the *lumped inelasticity* approach.

In the *equivalent strut* idealization proposed by Calderoni et al. (1987 and 1989), masonry panels are schematized as "variable geometry" struts, whose orientation and stiffness reproduce in mean the behaviour of the entire panel.

In the *non-linear beam* idealization, the masonry panels are schematized as shear-deformable beams with non-linear behaviour. Some models assume a variable stiffness depending on the dimension of the compressed sections (Braga and Dolce 1982). Others models consider a constant stiffness in the elastic phase, with increasing inelastic deformations once the failure domain has been achieved (Tomazevic 1978, Tomasevic and Weiss 1990, Bosiljkov et al. 2007). The main drawbacks of the above mentioned models is to model spandrels as infinitely resistant, while all non-linearities are concentrated in piers. This assumption is quite reliable for new masonry buildings, in which stiff r.c. floors and beams are present, but is far to be attended in historic masonry buildings characterized by wooden floors and traction non resisting lintels. Moreover, it is worth noting that, although “secondary elements”, spandrels significantly affect the boundary conditions of piers (i.e. fixed-fixed ends or cantilever) with great influence on the seismic capacity. Few simple attempts to introduce deformable spandrels have been made by Braga e Dolce (1982) and Fusier and Vignoli (1993). In recent years, an increasing attention is paid to the analysis of spandrels, from both experimental (Calderoni et al. 2007, Gattesco et al. 2008, Graziotti et al. 2009, Dazio and Beyer 2010,) and theoretical point of view (Cattari 2007, Calderoni et al. 2007, Cattari and Lagomarsino 2008, Milani et al. 2009). Models illustrated in the following take into account both spandrel and pier elements. According to *non-linear beam* idealization, in TREMURI program, developed by Galasco et al. 2009, a simplified non-linear element has been implemented in which masonry-panel’s behaviour is given by a bi-linear relation with cut-off in strength and stiffness decay in non-linear phase.

In the *lumped inelasticity* approach, linear shear-deformable beams are connected by inelastic links in which damage, cracking, sliding and rotations are concentrated (Roca et al. 2005, Penelis 2006). In this class also the well-known SAM method proposed by Magenes e Della Fontana (1998) may be included. Developed for incremental static non-linear analysis of multi-storied walls (Magenes 2000) and successively extended to 3D problems (Magenes et al. 2001), it uses a two-node frame element formulated on a phenomenological basis. The procedure considers a linear elastic behaviour until the yield limit, which is defined in terms of global forces based on the shear and flexural resistance criteria. Plastic rotations and sliding occur beyond this limit, until the deformation limits, which depend on the failure mechanism, are attained.

With respect to the *monodimensional* models above introduced, it seems important noting that the non-linear response of mason panels is in most cases directly faced in terms of global stiffness, strength and ultimate displacement capacity by assuming a proper shear-drift relationship (commonly by assuming a relationship bilinear without hardening and residual capacity) with a very rough description of the hysteretic behaviour. Thus their use seems particularly suitable to perform monotonic non-linear static analyses. In

DRAFT

particular, regarding the strength evaluation, reference is usually made to simplified criteria proposed in the literature and norms aimed to interpreter the main failure modes which may occur in masonry panels (Rocking, Crushing, Bed Joint Sliding, Diagonal Cracking). These criteria are based on the approximate evaluation of the local/mean stress state produced by the applied forces on predefined points/sections of the panel, assessing then its admissibility with reference to the limit strength domain of the constituent material, usually idealised through simple schematizations based on few mechanical parameters (e.g. the compressive strength of the masonry, the diagonal tensile strength of masonry, the tensile strength of block, the parameters characterizing the mortar joints, that is the cohesion and the friction coefficient respectively). The reliability of this approach has recently been assessed in Calderini et al. (2009). Particular caution has to be paid on the use of these criteria in case of spandrel elements. Indeed, common practice is to adopt the same failure criteria formulated for piers, assuming spandrel behaviour as that of a pier rotated by 90°; only few norms propose some specifications for these elements (FEMA 306, NTC). Recently, as above mentioned, an increasing attention is paid to spandrels leading to proposals of specific strength criteria (Cattari and Lagomarsino 2008, Milani et al. 2009).

In the class of *bidimensional* models, in more general terms (thus not strictly related to the meaning usually attributed to the “bidimensional” term) reference is made to both: models aimed to introduce the "no tension" condition; more accurate models aimed to provide a synthetic mechanical description of main deformation, damage and dissipation mechanisms of panels. In most of cases, these latter models are more suitable to perform not only non-linear static analyses (both monotonic and cyclic) but also non-linear dynamic analyses.

With reference to models which introduce the implementation of the "no tension" condition, it is carried out in various ways: modifying the element's geometry (adaptive geometry) with the scope of excluding regions in tension (D'Asdia and Viskovic 1994 - Figure 4.61) or introducing a suitable formulation of the stress field within the panel (Braga and Liberatore 1990).

DRAFT

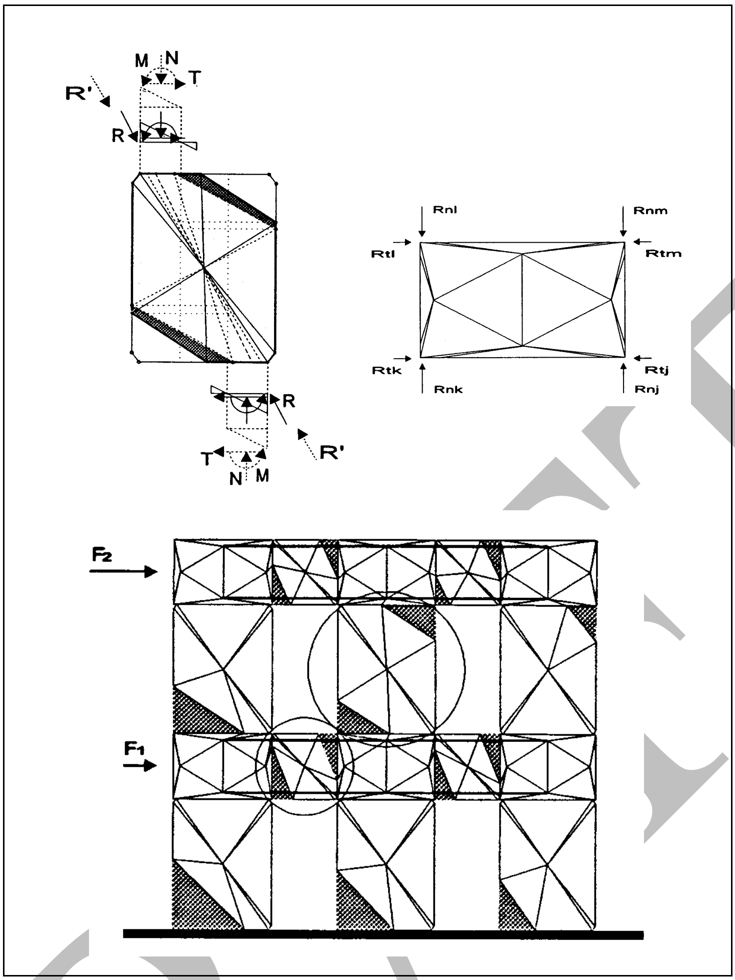


Figure 4.61. Model proposed in D'Asdia and Viskovic (1994).

The non-linear macro-element model illustrated in Galasco et al. (2004) and Penna et al. (2004) (which is implemented in the research version of TREMURI Program) permits, with a limited number of degrees of freedom (8), to represent the two main in-plane masonry failure modes, bending-rocking and shear-sliding (with friction) mechanisms, on the basis of mechanical assumptions. This model considers, by means of internal variables, the opening of tensile cracks at the corners (due to flexure) and the shear-sliding damage evolution, which controls the strength deterioration (softening) and the stiffness degradation. Figure 4.62 shows the three sub-structures in which the element is divided: the two layers, inferior and superior, in which the bending and axial effects are concentrated; the central part, that suffers shear-deformations and presents no axial or bending deformations. The shear model adopted in this element (originally proposed in Gambarotta and Lagomarsino 1996, Brencich et al. 1998) is a macroscopic representation of a continuous model (Gambarotta and Lagomarsino 1997), in which the parameters are directly correlated to the mechanical properties of the masonry elements. These parameters should be considered as representative of an average behaviour, but not as local parameters of the mortar joints. In addition to the original formulation previously described, the element used in the Tremuri program keeps also into account the effect (especially in bending-rocking mechanisms) of the limited compressive strength of masonry (Penna

DRAFT

2002). Toe crushing effect is modelled by means of phenomenological non-linear constitutive law with stiffness deterioration in compression.

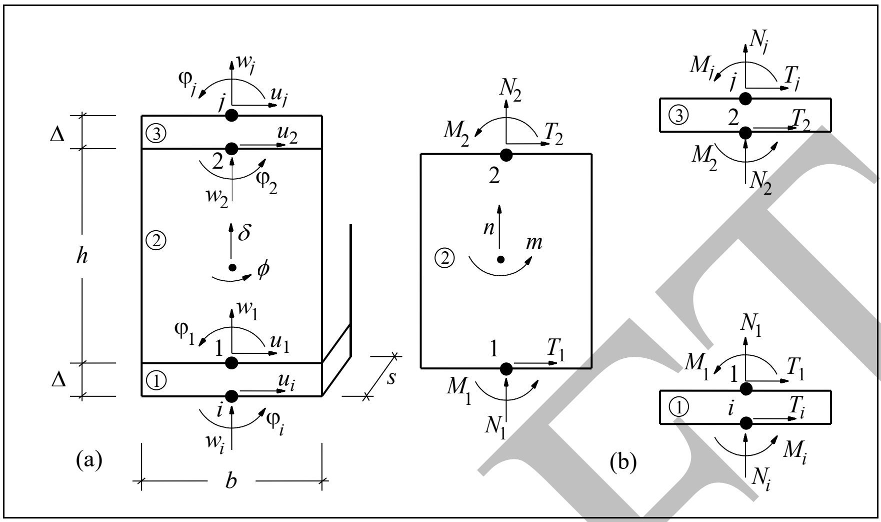

Δ
h
Δ
φ_j
w_j
u_j
j
③
u_2
2
φ_2
w_2
②
δ
ϕ
φ_1
w_1
u_1
1
①
u_i
i
φ_i
w_i
s
(a)
b
N_2
M_2
T_2
2
n
m
1
T_1
M_1
N_1
N_j
M_j
T_j
2
T_2
M_2
N_2
N_1
M_1
T_1
i
T_i
M_i
N_i
(b)

Figure 4.62. Model adopted in research version of Tremuri Program (Galasco et al. 2009).

DRAFT

Belmouden and Lestuzzi (2007) proposed a models based on a spread non linearity approach in which each pier and spandrel is discretized into a series of slices, which cross-sections are considered as homogeneous. The structural element behaviour is monitored at the centre of the slices while bending moments are evaluated at slice ends.

The element proposed recently in Chen et al. (2008) involves placing nonlinear shear springs in series with rotational springs to simulate both shear and flexure response (Figure 4.63). In particular, the model includes an axial spring, three shear springs, and two rotational springs to simulate the axial, bed joint sliding, diagonal tension, and rocking/toe crushing failure modes. Flexural behaviour is modelled by nonlinear rotational springs located at either end of the macroelement. The properties of the springs are based on the moment-curvature response of the top and bottom sections of the pier, which are established through the use of a fibre model.

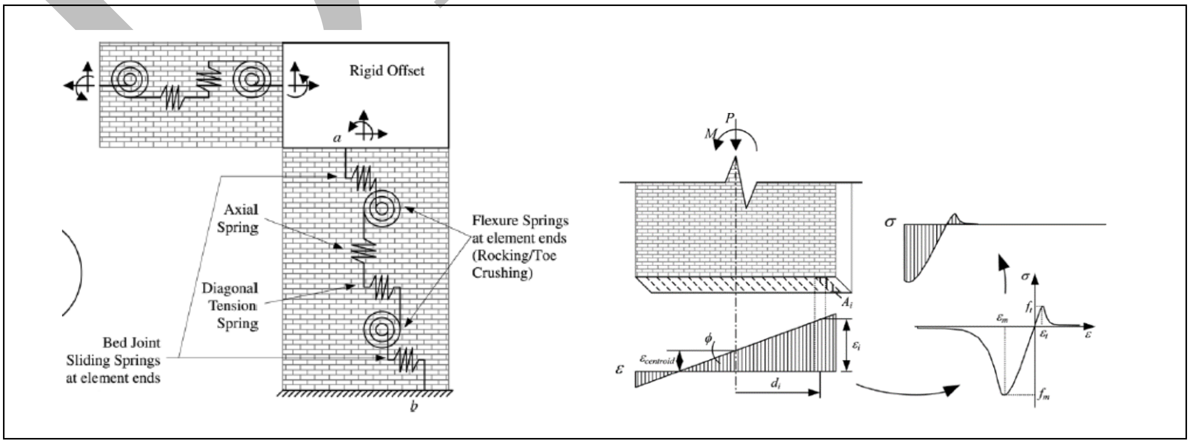

Rigid Offset
a
Axial Spring
Diagonal Tension Spring
Bed Joint Sliding Springs at element ends
Flexure Springs at element ends (Rocking/Toe Crushing)
b
P
M
A_i
ε
ε_centroid
ϕ
ε_i
d_i
σ
σ
f_t
ε_m
ε_t
ε
f_m

Figure 4.63. Model proposed in Chen et al. (2008).

Finally, recent modelling strategies based on assemblies of rigid bodies and non-linear springs (Caliò et al. 2007 – see Figure 4.64, Casolo and Peña 2007) present intermediate capabilities between Distinct Element and macro-element modelling. In case of model proposed by Caliò et al. (2007), each panel may be modelled by recurring to different mesh solutions (it means that each panel may be discretized by one or more elements). Recently, in Caliò et al. (2010) an extension of this element aimed to simulate the response of curved elements (like as the case of vaults) has been proposed.

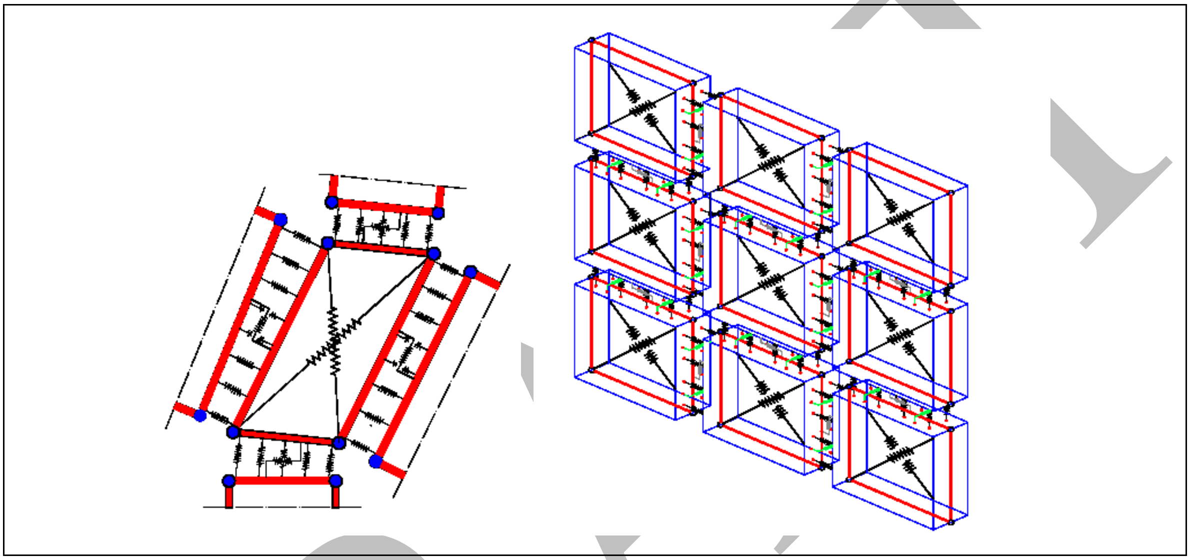


Figure 4.64. The model proposed in Caliò et al. (2007).

Finally, with reference to all the model afore mentioned, it seems useful noting that in almost all the cases only the in-plane response of masonry panels is simulated. Only in few cases, the out-of-plane response is also considered (Magenes and Della Fontana 1998, Caliò et al. 2007) even with different degree of accuracy (as an example by considering only the contribution to the stiffness). This hypothesis is consistent with the main use of these models in the context of the seismic assessment.

3.3.4 Interface models

In discrete interface models, masonry is modelled as a discrete system of elements (blocks, joints and/or interfaces).

Among them, the approaches based on limit analysis have been widely used; see for instance the original contribution of Livesley (1978) and the subsequent papers by Baggio and Trovalusci (1993, 1998, 2000), Gilbert and Melbourne (1994), Ferris and Tin-Loi (2001), Orduna and Lourenço (2005), Gilbert et al. (2006). In these papers, masonry is viewed as a discrete system where the blocks are considered as rigid and infinitely resistant bodies interacting with non-linear frictional joints. This approach seems reasonable for a wide class of historical masonry, in which the non-linear response is driven by the behaviour of the joints. Another model of rigid blocks connected by elastic constraints was proposed by Briccoli e Bati et al. (1995, 1999). In

DRAFT

this case, the joints are represented by a set of normal springs, while a shear spring is used to govern the behaviour in the shear direction.

The approaches based on the Distinct Element Method (Cundall 1976) involve soft contact formulation (deformable contact approach), i.e. contact stresses are function of relative block displacements, given the joint normal and shear stiffness properties (a normal interpenetration is needed to recognize contact between two different bodies). They may be used as well for static and dynamic analyses of masonry, through the explicit integration of the dynamic equations of motion (Pagnoni 1994, Pagnoni and Vanzi 1995, Azevedo et al. 2000, De Felice and Giannini 2001, Lemos 2007). Most of these approaches concern periodic masonry, made with parallelepiped blocks with horizontal and vertical joints, while only a few contributions have been devoted to irregular masonry, made with non squared blocks assembled in a non-periodic texture (see for example the research on the analysis of the ancient Sardinian Nuraghe structures by Roberti and Spina 2001 – Figure 4.65).

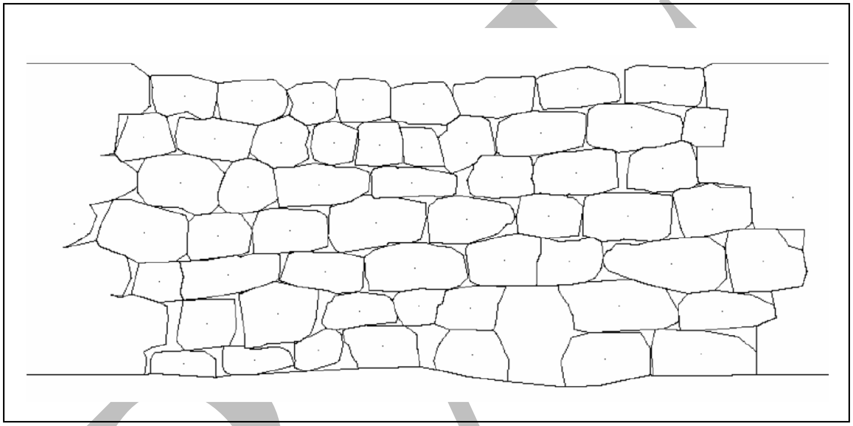


Figure 4.65. Modelling of irregular blocks (Roberti and Spina 2001).

A contact dynamic method, proposed by Jean (1995) and Jean and Acary (1998) differs in some respects from the typical DE treatment of contacts illustrated previously; unilateral contact conditions are assumed, with no block overlap (rigid contact approach).

Several authors developed models that combine features of the DE and FE methods, in order to take into account the deformability of blocks. Munjiza et al. (1995) developed a method for the simulation of fracturing problems considering deformable blocks that may split and separate during the analysis. Mamaghani et al. (1999) used a fixed contact system with a small deformation framework and finite deformations concentrated in contact elements. Contacts, discontinuities and interfaces were considered as bands with a finite thickness. The contact element was a two-nodded element having normal and shear stiffness. The method was applied to the stability analysis of different masonry structures. The failure mode of a masonry arch is shown in Fig. 4.66.

DRAFT

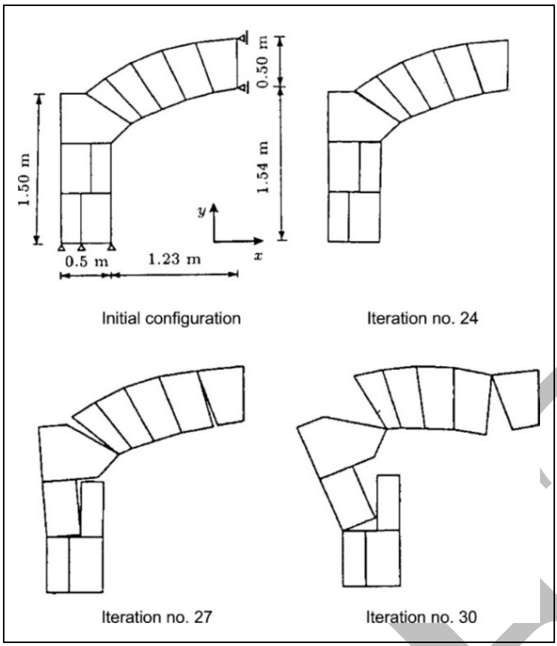


Figure 4.66. Failure mode of a masonry arch (Mamaghani et al.1999).

Another type of model is the Discontinuous Deformation Analysis (DDA) developed by Shi and Goodman (1988) firstly for rock engineering analysis and subsequently applied to masonry by Ma et al. (1996) and Bicanic et al. (2001). Blocks are considered deformable but in state of uniform stress and strain. Contact are assumed to be rigid and no penetration between blocks is permitted.

Finally, 2D or 3D Finite Element models allow a more refined representation of the state of stress and deformation of single components in a composite and complex system, such as masonry. However, the application of such techniques involves substantial computational time, especially with non-linear models (Page 1978, Lofti and Shing 1997, Lourenço and Rots 1997, Giambanco et al. 2001 – see Figure 4.67); for this reason, do not find extensive applications in real practice.

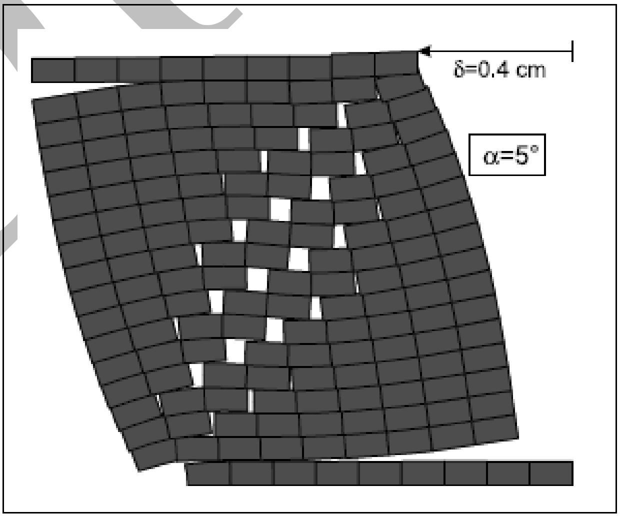


Figure 4.67. Finite element modeling of shear wall by a discrete approach (from Giambanco et al. 2001).

DRAFT

3.3.5 Macro-blocks models

Various studies highlighted the propensity of historical masonry structures to develop damage modes involving the formation of macro-blocks. The identification of macro-blocks shape in masonry structures derives from the observation of the main damage patterns in post-earthquake surveys.

The macro-blocks approach is widely used to perform the Limit Analysis of structures or the dynamic analyses of simple single or multi-body systems.

In the field of Limit Analysis, several works refer to Heyman's model (1966, 1982, 1995), whose main assumptions are: infinite compressive strength, no tensile strength and no sliding permitted. His work highlighted that the stability of masonry is more important than stress and the governing behaviour depend on geometry rather than material properties (the elastic calculation of stresses are not relevant and elastic deformations are omitted). Simple macro-blocks analyses consider the mechanics of simple arches or portals for both static (Heyman 1982, Ochsendorf et al. 2004 –Fiure 4.68) and seismic loads (Clemente and Raithel 1998, Ochsendorf 2002, Giordano et al. 2004 - Figure 4.69, De Luca et al. 2004). Macro-block modelling strategies for two stories shear walls with two openings were proposed by Orduna (2003) and Orduna et al. (2006). Some authors presented a method to evaluate the limit strength of masonry buildings modelling separately piers and spandrels (Como et al. 1988, Abruzzese et al. 1992, Orduna et al. 2006 – Figure 4.70). Based on the observation of real seismic failure modes of historical and traditional buildings, Giuffré (1989 and 1991 - Figure 4.71, see also Giuffrè and Carocci 1993 - Figure 4.72 - and Carocci 2001), proposed an approach for the study of the seismic vulnerability of more complex aggregates.

DRAFT

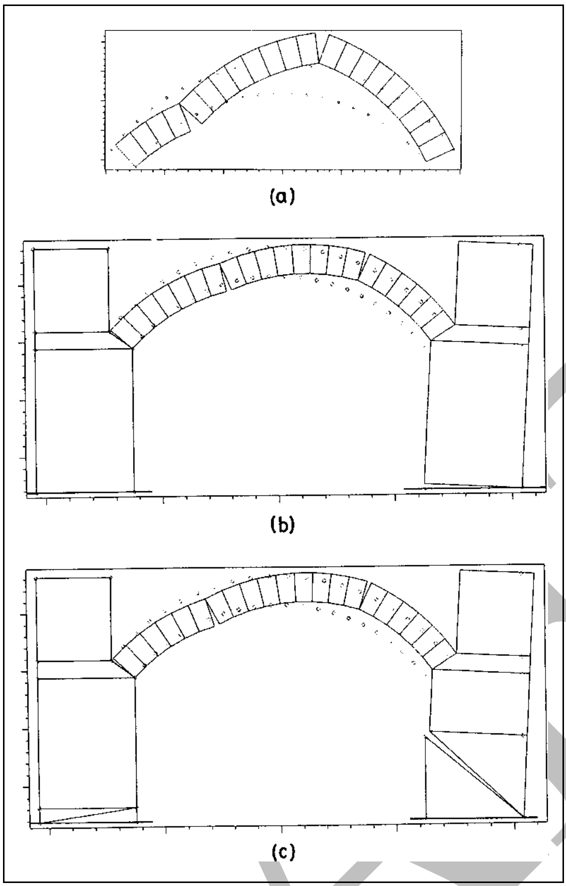


Figure 4.68. Collapse mechanism: a) arch; b) arch on rigid abutments; c) arch on fractured abutments (Ochsendorf et al. 2004).

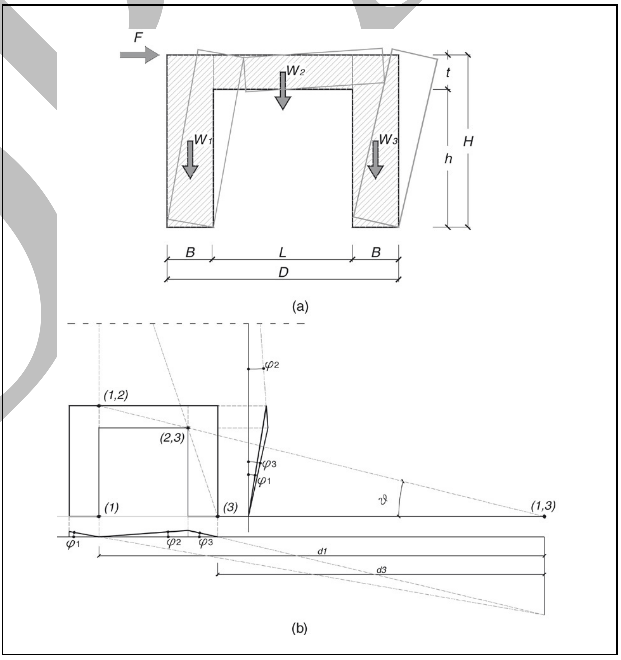

F
W2
W1
W3
t
H
h
B
L
B
D
(a)
φ2
(1,2)
(2,3)
φ3
φ1
ϑ
(1)
(3)
(1,3)
φ1
φ2
φ3
d1
d3
(b)

Figure 4.69. Collapse mechanism of a portal frame and kinematic chain (Giordano et al. 2004).

DRAFT

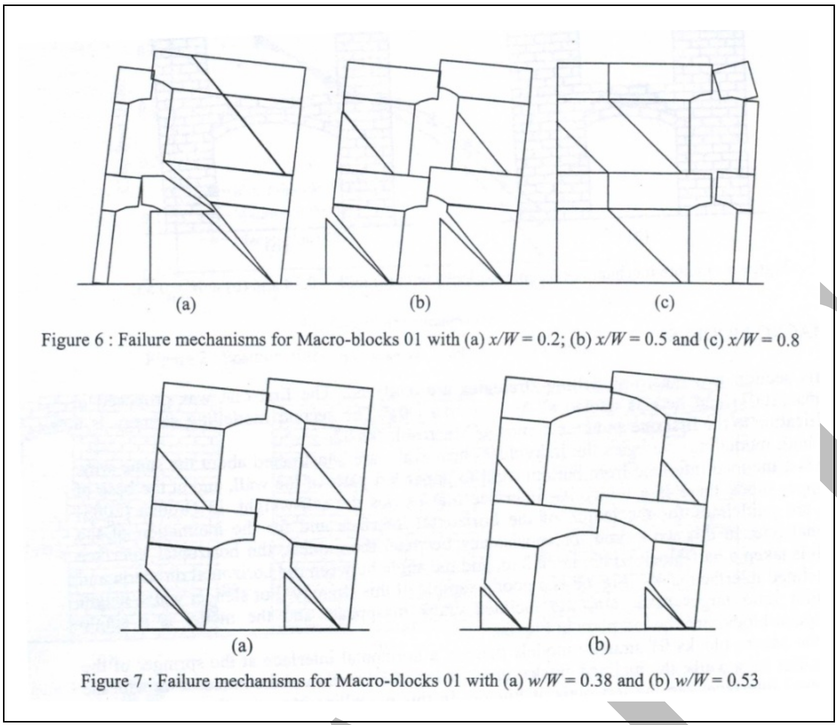


Figure 6 : Failure mechanisms for Macro-blocks 01 with (a) $x/W = 0.2$; (b) $x/W = 0.5$ and (c) $x/W = 0.8$

Figure 7 : Failure mechanisms for Macro-blocks 01 with (a) $w/W = 0.38$ and (b) $w/W = 0.53$

Figure 4.70. Macro-Block model of a two stories shear wall with two openings (Orduna et al. 2006).

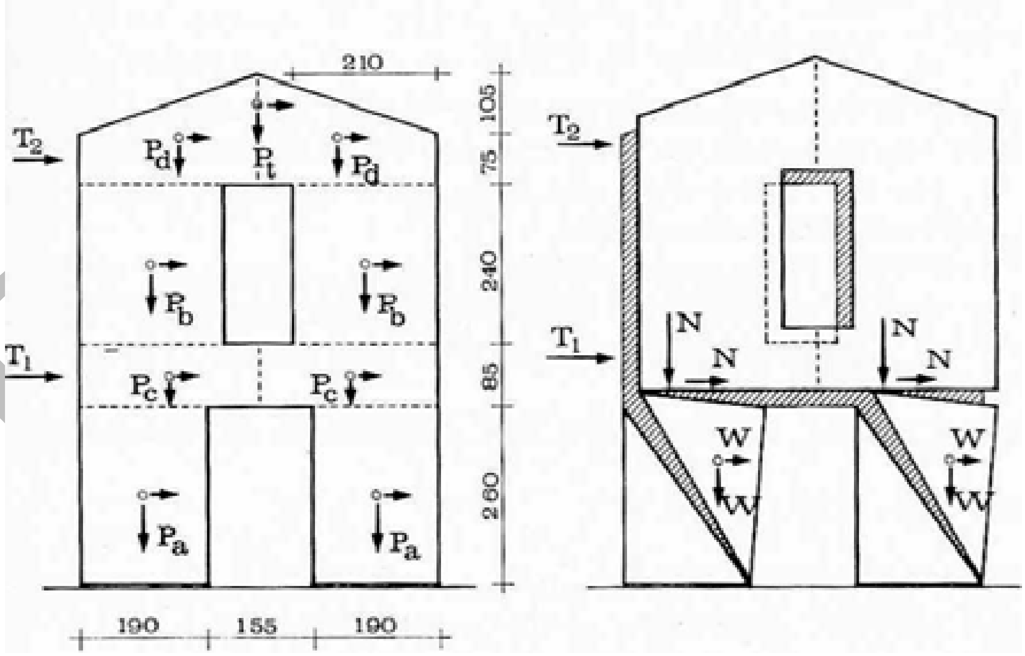

210
T2
Pd
Pt
Pd
Pb
Pb
T1
Pc
Pc
Pa
Pa
105
75
240
85
260
190
155
190
T2
T1
N
N
N
N
W
W
W
W

Figure 4.71. Identification of macro block on a masonry building façade (Giuffrè 1991).

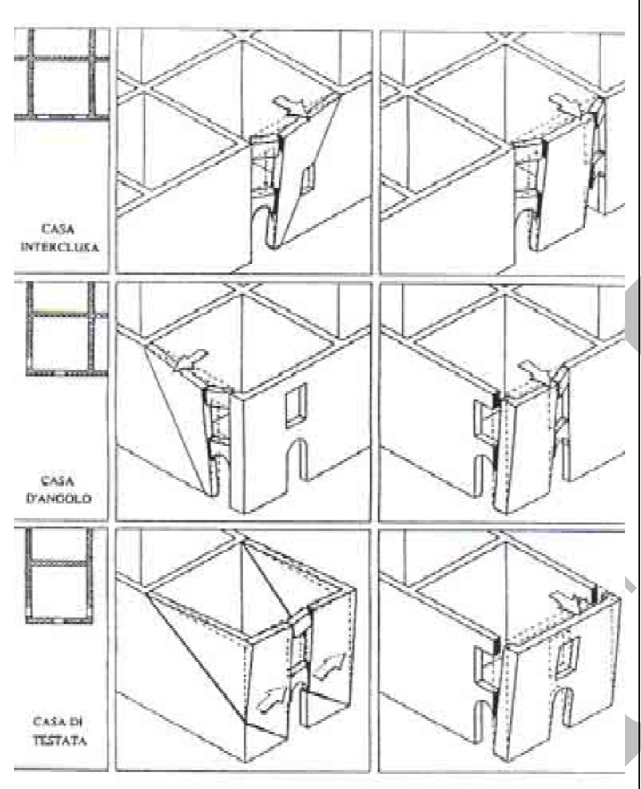


Figure 4.72. Collapse mechanism of buildings of historical centres (Giuffré and Carocci 1993).

Some authors worked on the frictional behaviour of the interface, considering that especially for historic buildings the quality of the contact surfaces or binding materials might be deteriorated so as to substantially reduce the original friction coefficient and to allow sliding mechanism (Casapulla et al. 1998, Casapulla and D'Ayala 2001 and 2006; D'Ayala and Speranza 2003, D'Ayala and Tomasoni 2008).

In the field of dynamic analysis, rigid-block models provide a suitable framework for understanding the behaviour of structures under seismic actions. In this case, the problem is primarily concerned with Rocking Motion (RM) dynamics. One of the first work is that of Housner (1963) which analysed rocking motion of a rigid block on a rigid base subjected to ground motion. Introducing a measure of the energy loss due to impact (damping of the system), known as coefficient of restitution, it takes in account that bodies are not fully rigid. Housner's coefficient, depending on the slenderness ratio of the block, was then adopted by Giannini (1984), who also introduced the dependence on the application point of the resulting impulsive forces due to impact. Assuming a model with a slightly rounded block, Capecchi et al. (1996) took in account the possibility to have imperfections in the contacts between foundation and block (Figure 4.73). Some researchers considered the possibility to have sliding, bouncing and rocking behaviour of the body (Ishiyama 1982, Sinopoli 1989, Augusti and Sinopoli 1992, Jeong et al. 2003, Nozaki et al. 2009), blocks damage (Augusti and Ciampoli 1993). Prieto and Lourenco (2005) proposed a new formulation of differential equation describing rocking motion. Lipscombe and Pellegrino (1993) focused the attention on short blocks (whose impact response is difficult to predict with simple rocking models), developing a two and three-dimensional bouncing model. Sinopoli and Sepe (1993) investigated the behaviour of a three block frame structure

DRAFT

subjected to horizontal ground acceleration. Similarly, Spanos et al. (2001) focused on the dynamic behaviour of structures consisting of two rigid blocks; assuming rigid foundation, large friction to prevent sliding, and point contact during a perfectly plastic impact, the only possible response mechanism under base excitation is rocking about the corners of the blocks (Figure 4.74).

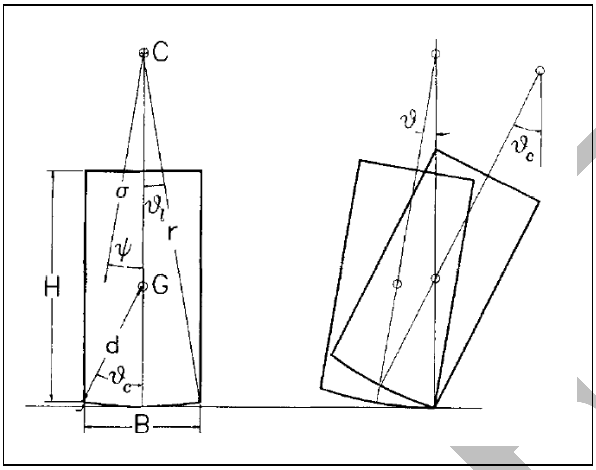

C
σ
ϑ_l
r
ψ
G
H
d
ϑ_c
B
ϑ
ϑ_c

Figure 4.73. Rocking of a rigid body with rounded base (Capecchi et al. 1994)

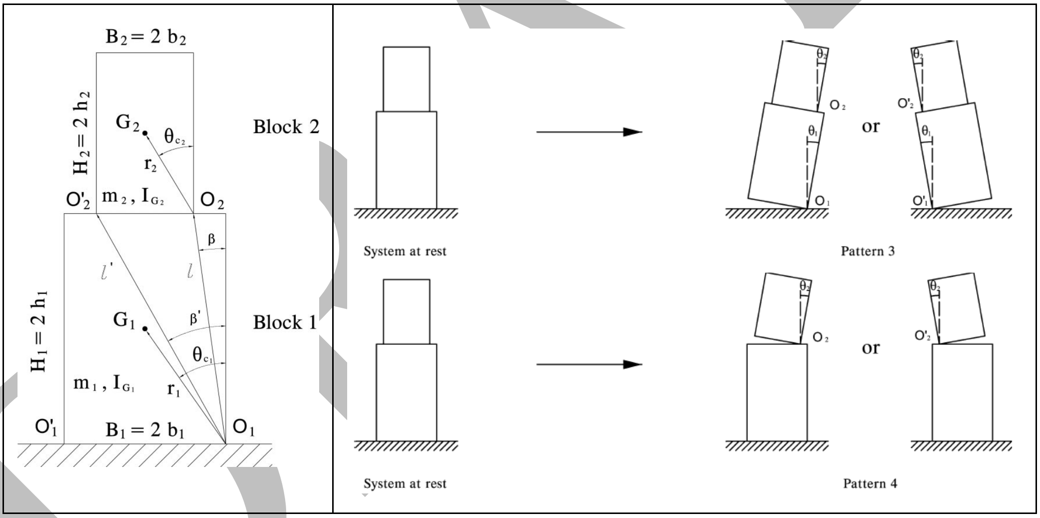

B₂ = 2 b₂
H₂ = 2 h₂
G₂
θc₂
r₂
m₂ , IG₂
O'₂
O₂
β
l'
l
G₁
β'
θc₁
H₁ = 2 h₁
m₁ , IG₁
r₁
O'₁
B₁ = 2 b₁
O₁
Block 2
Block 1
System at rest
θ₂
θ₁
O₂
O₁
or
O'₂
O'₁
Pattern 3
System at rest
Pattern 4

Figure 4.74. Geometric model of two stacked rigid block and example of possible rocking mechanism (Spanos et al. 2001).

The assumption of rigid body behaviour can be considered realistic only for low values of vertical forces. When subjected to high compressive forces the individual block of masonry walls can assume a "semi-rigid" behaviour, deforming significantly. A simplified procedure to evaluate the out-of-plane seismic response of URM walls is carried out by several researchers (Priestley 1985, Doherty et al. 2002, Sorrentino et al. 2008), considering energy dissipation due to geometric imperfections, deformability of blocks, etc., and considering wall models with different boundary conditions.

DRAFT

Oppenheim (1992) was the first to apply a dynamic analysis to the masonry arch (Figure 4.75). He analysed the dynamic response of an arch considered as rigid body four-link mechanism under a base acceleration impulse. The impulse magnitude-duration failure domain was determined assuming failure during a single cycle of response in order to avoid the problems associated to impact. Clemente (1998) followed up on Oppenheim's work, by analysing free vibration and response to sinusoidal base acceleration. The analysis was extended to several cycles of response, but the dissipation of energy due to impact was not considered. Their analysis also does not include the possibility of sliding or more complicated arch hinging mechanisms. De Lorenzis et al. (2007) proposed then an analytical model which follows Oppenheim's approach but also tackles the impact problem and hence predict the behaviour of the arch during subsequent half cycles of motion.

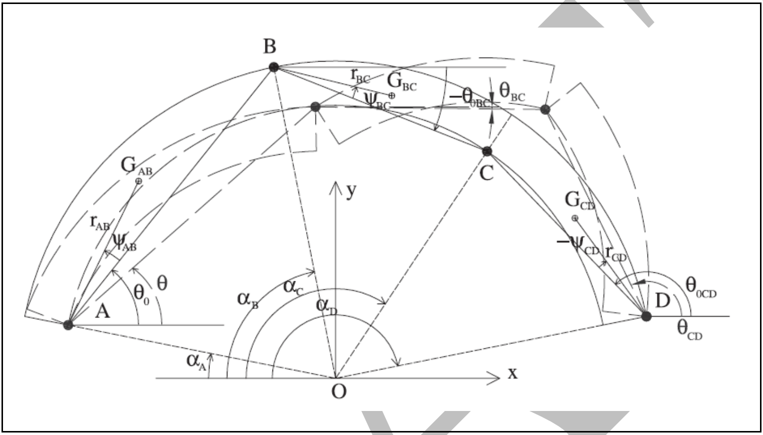


Figure 4.75. The arch as a four-link mechanism during the first half cycle of motion (Oppenheim 1992).

3.4 *Distinctive features and tricks related to the different types of models*

3.4.1 *Modelling by structural elements models (SEM)*

Structural element modelling (SEM) strategies are based on the identification of macroscopic structural elements, defined from a geometrical and cinematic point of view through finite elements (solid, shell or frame) and described from a static point of view through their internal generalized forces.

In the field of structural element models, "equivalent frame" models are the most widely diffused. They consider the walls as an idealized frame, in which deformable elements (piers and spandrels) connect rigid nodes (parts of the wall which are not usually subjected to damage). Piers are the principal vertical resistant elements for both dead and seismic loads; spandrels, which are intended to be those parts of walls between two vertically-aligned openings, are secondary horizontal elements, coupling piers in the case of seismic loads. It is worth noting that, although "secondary elements", spandrels significantly affect the boundary conditions of piers (i.e. fixed-fixed ends or cantilever) with great influence on the seismic capacity.

Figure 4.76 summarizes the idea of the equivalent frame model (EFM) compared with other simplified modelling strategies (still ascribable to the SEM approach) that disregard the role of spandrels (e.g. the POR

DRAFT

Method, as originally proposed in Tomaževič 1978, or the "weak spandrel-strong piers" and "strong spandrel-weak piers" models, as proposed in FEMA 356 and 306 documents).

In particular, the idealisation of a "strong spandrels-weak piers" model (Model I in Figure 4.76) is based on the assumption piers crack first, thus preventing the failure of spandrels which can be then assumed as infinitely stiff portions, assuring a perfect coupling between piers. This corresponds to assuming a fixed-rotation boundary condition at the piers extremities and it is also known as "storey mechanism" (POR Method, SREMB Model). On the contrary, in case of the "weak spandrels-strong piers" (Model II in Figure 4.76), the hypothesis of both null strength and null stiffness of spandrels is adopted then assuming the piers as uncoupled (this corresponds to the cantilever idealisation). However, it is worth noting that in most cases it is correct to assume that horizontal displacement of the vertical structural elements is at least coupled at the floor levels by the presence of horizontal diaphragms.

Despite the advantage of adopting very simplified and manageable models, since they are based on an aprioristic choice, the following troublesome issues arise. First of all, it is conceivable that both of these limiting cases are inappropriate for certain walls, which may display both types of response in different regions or which can be involved in a different behaviour with the increase of nonlinear response. Moreover, it is not at all a foregone conclusion that the presence of certain constructive details (e.g. r.c. beams coupled to spandrels), not supported by a quantitative evaluation of their effectiveness, is sufficient to assure the achievement of the hypotheses which these simplified models are based on (Cattari and Lagomarsino 2009). On the contrary in case of the EFM Model, since both pier and spandrel elements are modelled, the transition through different boundary conditions is directly a consequence of the progressive damage of the elements. Actually, in some cases, the use of the EFM Model is regulated in norms, by defining the cases in which masonry spandrels may be taken into account as coupling beams in the structural model (Eurocode 8); these provisions mainly concern the bonding to the adjoining walls, the connection both to the floor tie beam and to the lintel. Moreover, the idealisation as an equivalent frame easily allows to introduce other structural elements, such as reinforced concrete beams or columns, together with the masonry ones. Thus, it appears particularly versatile to model also mixed structures (e.g. mixed masonry and reinforced concrete structures which are quite common in existing buildings). Moreover, the EFM approach usually requires a limited number of degrees of freedom, with a reasonable computational effort: thus it allows the analysis of complex 3D models of URM structures, obtained by assembling walls and considering only their in-plane contribution.

DRAFT

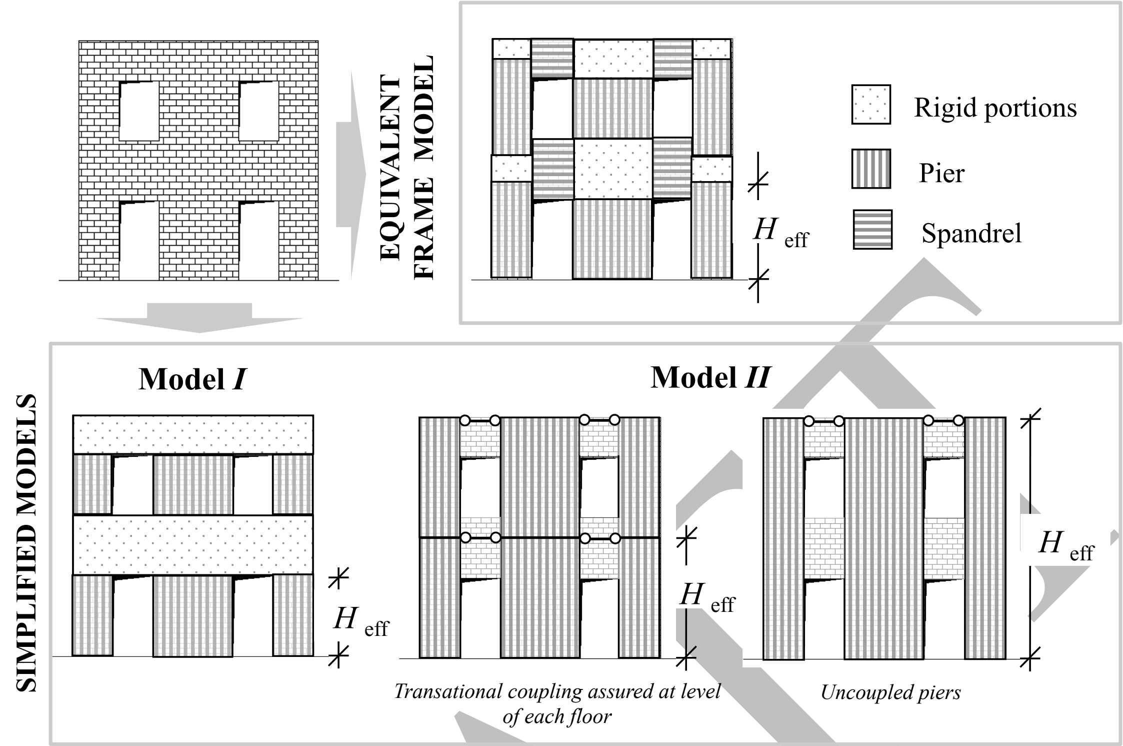


Figure 4.76. URM wall idealisation according to Simplified and Equivalent Frame Models ascribable to SEM strategy (adapted from Cattari and Lagomarsino 2009)

All the aforementioned issues highlight the technical advantage of EFM models by favouring this adoption with respect to other more simplified solutions; this is even more relevant in case of monumental buildings for which a reliable assessment provided by models is one of key tools to guarantee their preservation.

Thus, in the following the attention is focused only on EFM models. Within this context, a review of its use in engineering practice is proposed paying particular attention to some different solutions, proposed in the literature and adopted in some software, to apply the concepts of the equivalent frame model; in fact, although this approach is expressly advised by both international and national norms, the arbitrariness of its application may be quite wide.

Despite the general effectiveness of most of remarks presented, in many cases the solutions implemented in the TREMURI program are illustrated more in detail. This software has been originally developed and gradually improved at the University of Genoa, starting from 2001 (Galasco et al. 2004, Lagomarsino et al. 2012 and Lagomarsino et al. 2013) and subsequently also implemented in the commercial software 3Muri. Then, specific multi-linear constitutive laws aimed to describe the response of masonry panels until very severe damage levels and to simulate different hysteretic response, have been developed and added.

In particular, the following issues will be discussed: i) idealization of masonry walls as an equivalent frame (e.g. rules to define panels geometry); ii) modelling of non-linear response of structural elements (with particular attention to masonry panels); iii) diaphragms modelling (e.g. adequate definition of the stiffness properties) and assembling of 3D models (e.g. solutions to connect walls in corners); iv) modelling of foundation and soil. Finally, a summary of the geometrical and mechanical data necessary for the application of SEM model is illustrated.

DRAFT

3.4.1.1 Identification of piers and spandrels

The first step for modelling of the masonry wall as an equivalent frame is the identification of the main structural components, previously introduced as *piers* and *spandrels*.

For the identification of the geometry of pier and spandrel elements, conventional criteria are often assumed in literature, supported by the damage survey after earthquakes and experimental campaigns. However, a systematic parametrical analysis either numerical or experimental has never been performed in order to define rigorous criteria. Despite this, although the identification of masonry piers and spandrels may result rather trivial and easily automated in case of perforated walls with regularly distributed openings, it becomes more difficult and ambiguous when openings are irregularly arranged (Figure 4.77). In the following some possible criteria are examined.

Usually the criteria for the definition of the height of masonry piers are defined as a function of that of adjacent openings by assuming a conventional stress diffusion (e.g. by assuming a cone diffusion at 30° degrees, as proposed in Dolce 1991 see Figure 4.78). In Yi et al. (2006) it is proposed to define it as the height over which a compression strut is likely to develop at the steepest possible angle (i.e. assuming that cracks can develop either horizontally or at 45°, see Figure 4.78). In case of existing buildings, the pattern of pre-existing cracks should be taken into account in order to properly define the geometry of spandrels and piers.

DRAFT

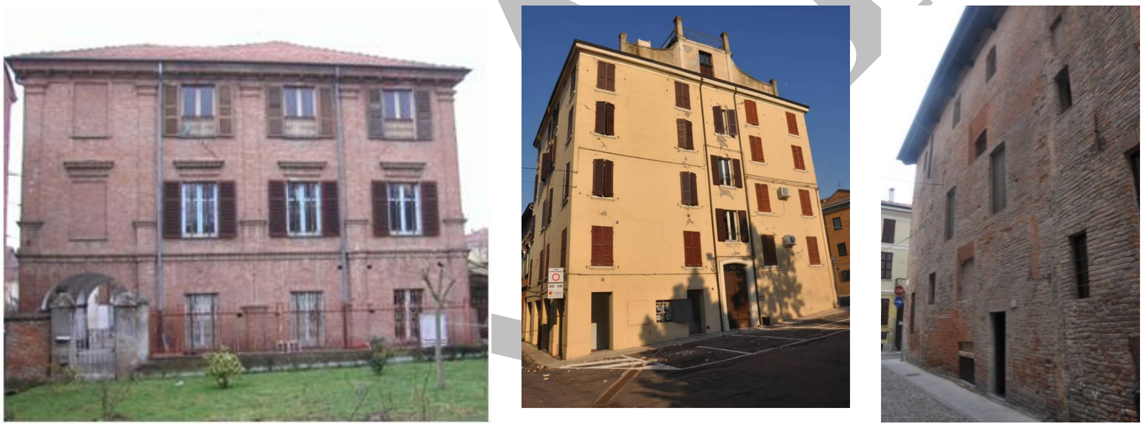


Figure 4.77. Examples of façades with both regularly and irregularly distributed openings

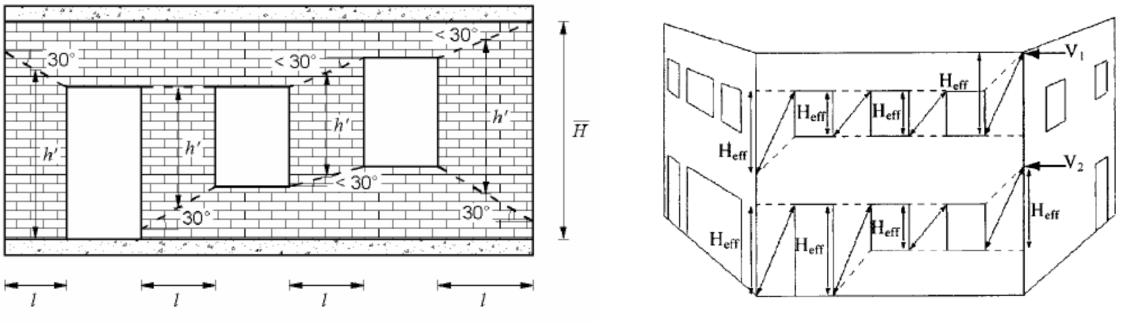


Figure 4.78. Examples of façades with both regularly and irregularly distributed openings Example of some criteria proposed in literature for the definition of pier height (on the left from Dolce (1989), on the right from Yi et al. (2006))

In the following some criteria that can be easily automated are discussed. Figure 4.79 summarizes the main steps of the frame idealization procedure in a regularly perforated masonry wall: from the identification of spandrels and piers (steps 1 and 2) to that of nodes (step 3).

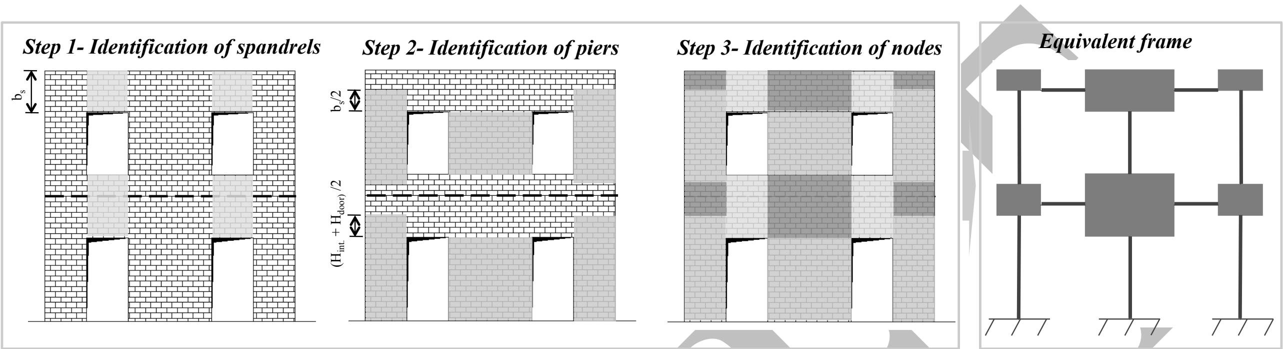

Step 1- Identification of spandrels
b_s
Step 2- Identification of piers
$b_s/2$
$(H_{int.} + H_{door})/2$
Step 3- Identification of nodes
Equivalent frame

Figure 4.79. Example of equivalent frame idealisation in case of regularly distributed openings (from Lagomarsino et al. 2013)

Spandrel beams (step 1) are defined on basis of the vertical alignment and overlap of openings: the length and the height are assumed equal to the distance and width (in case of full alignment) of the adjacent openings, respectively. Piers beams (step 2) are defined starting from the height of adjacent openings: when these latter are perfectly aligned, as the case of the internal pier shown in **Errore. L'origine riferimento non è stata trovata.**, the height is assumed equal to that of openings. For the definition of height of the external piers the possible development of inclined cracks from the opening corners (and/or from the lintel edges) has to be considered, as previously discussed. As possible approximated criteria, it can be assumed equal to the height of the adjacent opening or as the average of the interstorey height and the height of the opening. The height of the rigid nodes (step 3) is then generally defined as the average value of the height of the adjacent spandrels. To complete the frame idealization for the whole wall, such a calculation is done separately for each storey and each wall. It is worth noting that the application of such a criterion without any limitation to the cone diffusion angle may induce a significant overestimation of the effective in-plane aspect ratio of external piers in case of adjacent openings with a limited height and close to the wall edge. Actually, in these situations flexural failure modes are likely predicted for such slender piers, with possible underestimation of the lateral strength and overestimation of the deformation capacity. The presence of structural elements, such as lintels and r.c. tie-beams, can influence the effective height of masonry piers and, in principle, for irregularly distributed openings it should also vary depending on the direction of analysis.

Moreover, if present, infilled openings (as shown in Figure 4.80) could be idealized, on the safe side, as doors or windows, so neglecting the contribution of added masonry. This is justified by the difficulty to guarantee a full interlocking with the adjacent pre-existing masonry portions and the stress redistribution effects, which hardly may reproduce the original configuration without opening. As an alternative, reduced mechanical properties should be assigned to the corresponding infilled masonry portions. In case of not perfectly aligned openings, a possible choice is to conventionally assume a mean value for the height of spandrel elements as

DRAFT

a function of the overlapping part between the openings at the two levels (Figure 4.80); when no overlap is present or the opening lacks at all, it seems more appropriate (from the observed damage survey) to assume the portion of masonry as a rigid area (Figure 4.80). Definitely, further studies, based on both experimental testing and numerical research, should be performed in order to validate the capability of the presented procedure for different types of masonry layout.

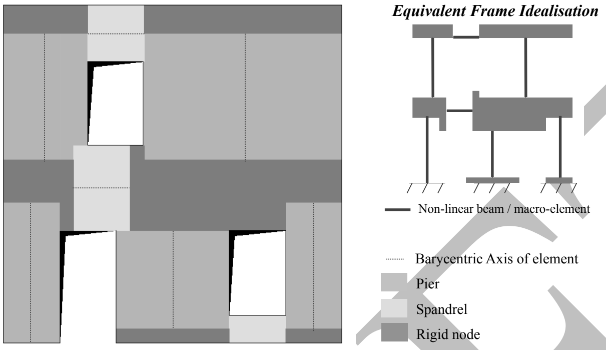

Equivalent Frame Idealisation
Non-linear beam / macro-element
Barycentric Axis of element
Pier
Spandrel
Rigid node

Figure 4.80. Example of equivalent frame idealisation in a case of irregularly distributed openings (Lagomarsino and Cattari 2009)

3.4.1.2 Modelling of structural elements

Once having idealised the masonry wall into an assembly of structural elements, the reliable prediction of its overall behaviour mainly depends on the proper interpretation of the single element response.

Several formulations, characterized by different degrees of accuracy, may be adopted either for masonry panels and other structural types. The possibility of modelling the non-linear response of structural elements other than masonry ones, such as reinforced concrete (RC), steel or wooden beams, is particularly useful for the analysis of new and existing buildings and to simulate subsequent possible transformations occurred in the examined asset.

As regard to masonry panels, either monodimensional (according to different idealizations like the *equivalent strut*, the *non-linear beam* or the *lumped inelasticity* approaches) and bidimensional structural elements may be adopted. Figure 4.81 synthetises the classification proposed.

DRAFT

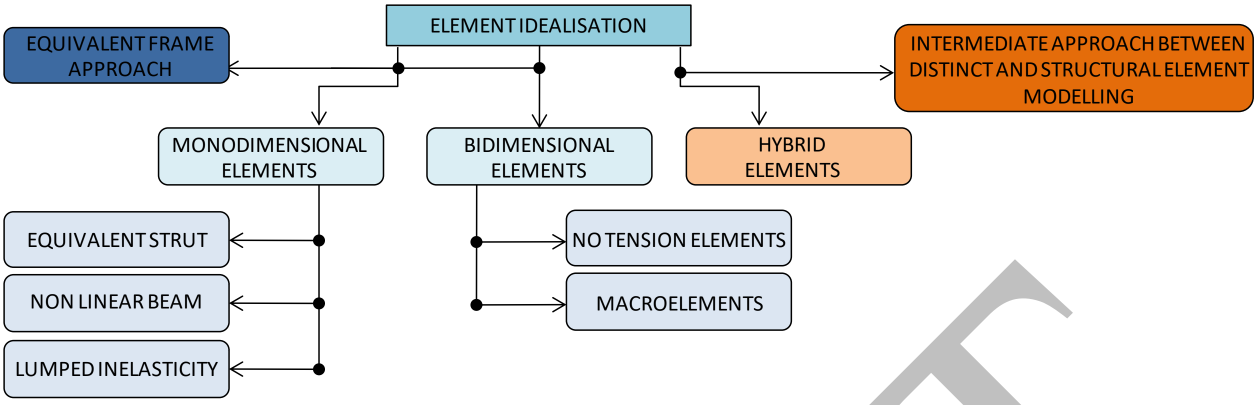

ELEMENT IDEALISATION
EQUIVALENT FRAME APPROACH
INTERMEDIATE APPROACH BETWEEN DISTINCT AND STRUCTURAL ELEMENT MODELLING
MONODIMENSIONAL ELEMENTS
BIDIMENSIONAL ELEMENTS
HYBRID ELEMENTS
EQUIVALENT STRUT
NON LINEAR BEAM
LUMPED INELASTICITY
NO TENSION ELEMENTS
MACROELEMENTS

Figure 4.81. Scheme of the main approaches adopted in the field of structural elements models.

In the following, the attention is focused only on the formulation based on non-linear beam elements. Thus, the element response is directly faced in terms of stiffness, strength and displacement capacity (corresponding to different damage levels) by assuming a proper force-displacement relationship and appropriate drift limits (or chord rotation limits in the case of RC elements). Despite some unavoidable approximations of the actual behaviour (e.g. related to the mechanical description of damage and dissipation mechanisms), this simplified formulation implies the following main advantages:

- it allows performing nonlinear static analyses with a reasonable computational effort, suitable also in engineering practice;
- it is based on few mechanical parameters that may be quite simply defined and related to results of standard tests.

Moreover, it has to be stressed this formulation is consistent with the recommendations included in several seismic norms (Eurocode 8, NTC 2008 and 2018, ASCE/SEI 41-06), since strength criteria defined for both bending and shear failure modes can be easily implemented and adopted to define the lateral strength of the different structural elements. Thus, it allows a wider diffusion also in the engineering practice.

Modelling of masonry elements

The specific characterization of the force-displacement relationship, aiming to describe the masonry panels behaviour, starts from the knowledge and interpretation of the different failure modes which may occur.

Observation of seismic damage to complex masonry walls, as well as laboratory experimental tests, have shown that a masonry panel subjected to in-plane loading may show two typical types of behaviour:

- *flexural behaviour*, that may be associated to the failure modes of *Rocking* (panel starts to behave as a nearly rigid body rotating about the toe) and *Crushing* (panel is progressively characterized by a widespread damage pattern, with sub-vertical cracks oriented towards the compressed corners);
- *shear behaviour*, that may be associated with the failure modes of *Diagonal Cracking* (panel usually develops cracks at its centre, that after propagate towards the corners) and *Shear Sliding* (failure is attained with sliding on a horizontal bed joint plane).

DRAFT

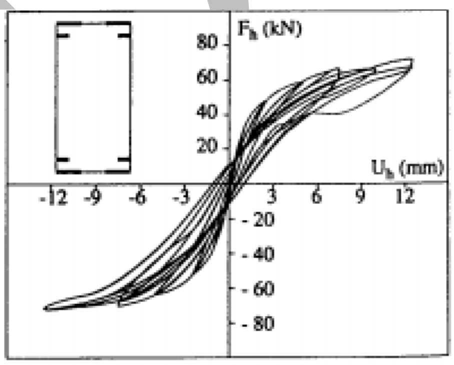
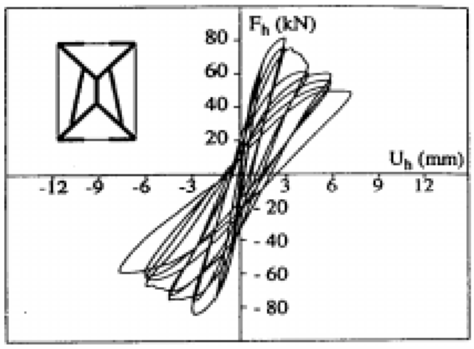
Despite this classification, it is evident that also mixed modes are possible and quite common. These different failure modes vary not only in the prevailing damage pattern and shear strength associated, but also in the hysteretic response.

Actually it is worth noting that this classification usually is implicitly referred to the pier element type. In fact, while many experimental researches related to the behaviour of piers have been carried out in the last decades (for example those described in Shah and Abrams 1992, Anthoine et al. 1995, Magenes and Calvi 1992, Bosiljkov 2003, Vasconcelos 2006, Lourenco 2005, Galasco et al. 2010), tests on spandrels are very limited and quite recent (Gattesco et al. 2008, Beyer and Dazio 2012a and 2012b, Graziotti et al. 2012, with reference only on full scale samples).

Indeed, the boundary conditions that characterize spandrel elements and the orientation of main mortar joints activated are very different from those of piers: as a consequence, relevant differences may be noticed. In particular, in case of *flexural behaviour*, due to low values of axial load, which usually characterize spandrel elements (especially in case of lack of tie-rods or r.c. beams), *Crushing* represents a very rare instance. Moreover, in case of the *shear behaviour*, due to the interlocking phenomena, *Sliding* failure (meant as sliding on a vertical bed joint plane at the end-sections) usually cannot occur.

As well known, the occurrence of these different failure modes depends on several parameters. In case of piers, they may be summarized as follows: the geometry; the boundary conditions; the axial load; the mechanical characteristics of the masonry constituents (mortar, blocks and interfaces); the masonry characteristics (block aspect ratio, in-plane and cross-section masonry pattern). Figure 4.82 shows some results of experimental campaigns performed on piers.

DRAFT

	Prevailing flexural response	Prevailing shear response
Anthoine et al. 1995 (brick		

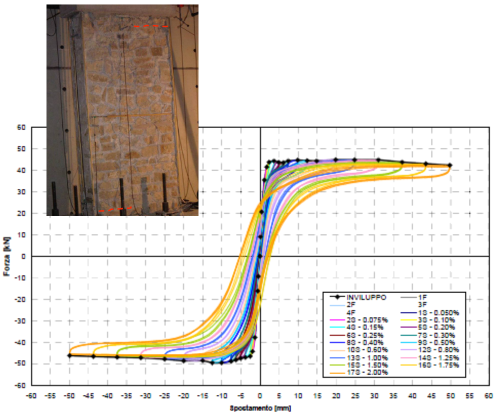
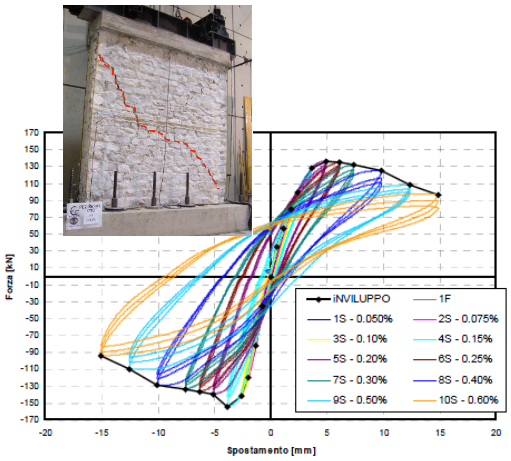
Galasco et. al. 2010 (double leaf stone masonry)		

Figure 4.82. Examples of results of experimental campaigns performed on masonry piers

In case of spandrels, as shown in the referenced experimental campaigns, some additional variables can play an important role, like the interlocking phenomena which can be originated at end-sections with the contiguous masonry portions, the type of lintels (materials, technologies, restraining systems), the interaction with other structural elements coupled to it (in particular if tensile resistant such as r.c. beams or steel tie-rods). Figure 4.83 shows some results of experimental campaigns performed on spandrels.

DRAFT

	Tensile resistant elements coupled to spandrel	
	Yes	No

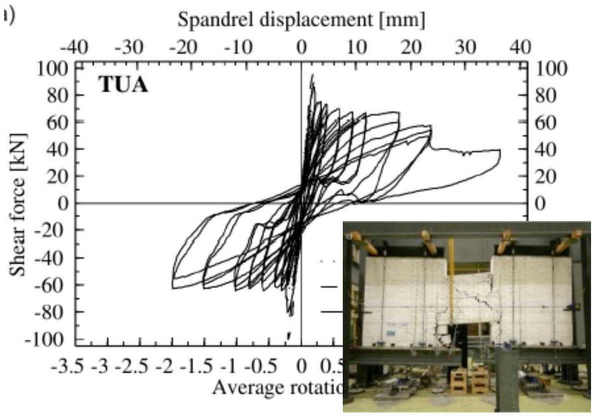
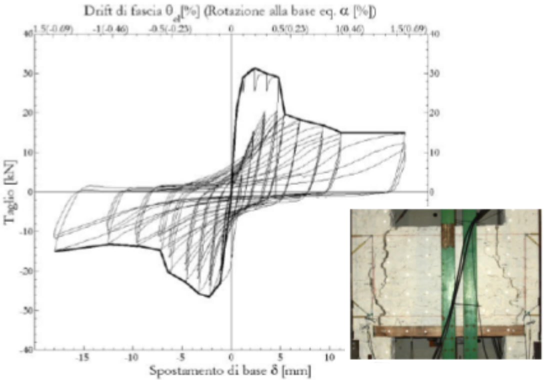
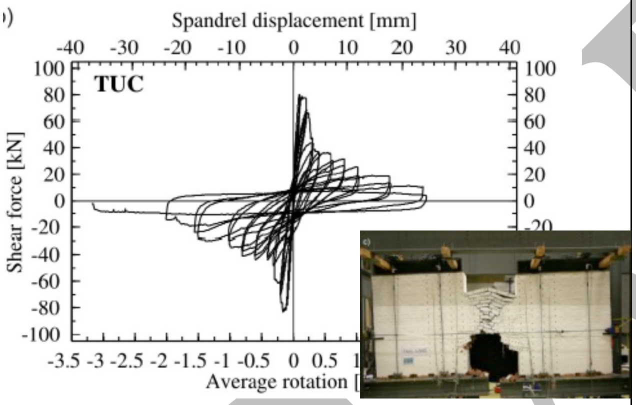
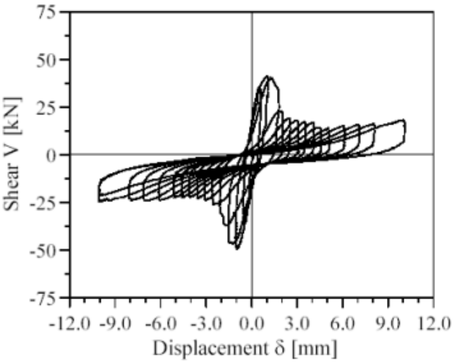
Architrave type	Wooden beam	 From Beyer and Dazio (2010) - *prevailing shear mode*	 From Graziotti et al. (2011) – *prevailing flexural mode*
	Masonry arch	 From Beyer and Dazio (2010) – *prevailing shear mode*	 From Gattesco et al. (2010) –*mixed flexural and shear mode*

Figure 4.83. Examples of results of experimental campaigns performed on masonry spandrels

The above introduced failure modes may be interpreted, in terms of resultant maximum shear, by some simplified strength criteria, based on mechanical or phenomenological hypotheses, which are proposed in literature and norms. Usually, they are based on the approximate evaluation of the local/mean stress state produced by the applied forces on predefined points/sections of the panel, assessing then its admissibility with reference to the limit strength domain of the constituent material, usually idealised through simple oversimplifications based on few mechanical parameters (as discussed in Calderini et al. 2009). As a function of the current value of the axial force (N) acting on the element, the minimum value – as predicted by the criteria adopted to model the *flexural* and *shear* responses, respectively – is usually assumed as reference. In addition, it is worth noting that, due to the application of horizontal load patterns, aimed to simulate seismic actions, the acting axial load changes from the initial value consequent to the vertical dead loads; moreover, due to redistribution phenomena associated with the progressing of nonlinear response, further variations may occur. As a consequence, it is evident how also the value of the corresponding shear strength varies in each panel during the nonlinear static analysis. Then, failure of the panel is usually defined through the definition of a maximum drift (δ_u) based on the prevailing failure mechanism occurred in the panel. Figure 4.84 schematically shows the abovementioned issues.

DRAFT

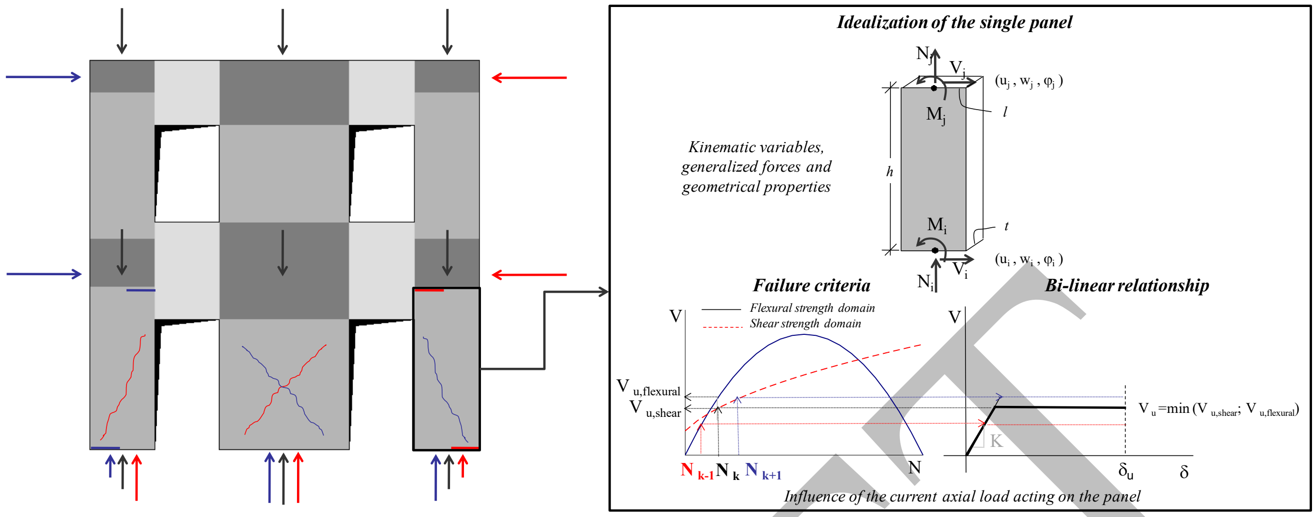

Idealization of the single panel
Kinematic variables, generalized forces and geometrical properties
N_j
V_j
M_j
(u_j, w_j, φ_j)
l
h
t
M_i
(u_i, w_i, φ_i)
N_i
V_i
Failure criteria
Flexural strength domain
Shear strength domain
V
$V_{u,flexural}$
$V_{u,shear}$
N_{k-1} N_k N_{k+1}
N
Bi-linear relationship
V
$V_u = \min(V_{u,shear}; V_{u,flexural})$
K
δ_u
δ
Influence of the current axial load acting on the panel

Figure 4.84. Sketch of the idealization of masonry pier response by adopting simplified strength criteria based on applied axial compression force

DRAFT

Usually, as suggested also by norms (e.g. Eurocode 8), non-linear beams aimed to describe masonry panels are modelled by assuming a bi-linear relation with cut-off in strength (without hardening). Also in the TREMURI code (and in the commercial version 3Muri) this solution has been originally adopted (Cattari and Lagomarsino 2009, Lagomarsino et al. 2013), by assuming a secant stiffness decay in the non-linear phase (for non-monotonic action). More recently, multi-linear constitutive laws aimed to describe the response of masonry panels until very severe damage levels and to simulate different hysteretic responses, have been developed and implemented in a new release of TREMURI.

Modelling of other structural elements (reinforce concrete, steel and wooden elements)

The possibility to model both masonry and other structural element (r.c., steel or wooden elements) with non linear behaviour is very important to guarantee a more reliable assessment. Despite this, it is worth noting that although there is a well-established background focused on the non-linear analysis of masonry structures and RC frames (also in the case of infilled frames), the set of numerical and experimental instruments for the study of their interaction effects is quite limited.

In the case of TREMURI program, nonlinear RC elements, modelled as 2D or 3D elements in the case of beams or columns and walls, respectively, are idealized by assuming elastic-perfectly plastic hinges concentrated at the ends of the element. The choice of this simplified concentrated plasticity model, with respect to more accurate ones like as the fibre approach, is justified by the will to assume a computational burden comparable to that of masonry elements and a similar level of accuracy. The initial elastic branch, similarly to masonry elements, is directly determined by the stiffness contributions in terms of shear and flexural behaviour by neglecting that offered by reinforcement. The reduction of stiffness due to cracking phenomena may be taken into account, analogously to masonry elements, by the ⍰ coefficient (e.g. assumed as proposed in Priestley 2003), kept constant during the analysis. Shear and compressive/tensile failures are assumed as brittle failures while combined axial-bending moment, modelled by plastic hinges at the end of element, are regarded as ductile failure. Further details on RC modelling elements may be found in Cattari and Lagomarsino (2013).

Also steel and wooden beams or tie-rods may be modelled. Similarly, to RC elements, steel and wooden beams are idealized by assuming elastic-perfectly plastic hinges, concentrated at the ends of the element. Obviously the strength criteria adopted as reference are modified according to these different materials and checks on the ultimate deformation capacity are not included. Tie-rods are idealized as non-compressive spar elements with the possibility to assign also an initial strain.

3.4.1.3 3D assembling of masonry walls

Starting from the equivalent frame modelling of each wall, complete 3D models may be assembled. The 3-dimensional modelling of whole URM buildings usually starts from the following basic hypotheses: a) the bearing structure, both referring to vertical and horizontal loads, is identified, inside the construction, with walls and floors (or vaults); b) the walls are the bearing elements, while floors are the elements which the sharing of vertical loads and the horizontal actions distribution between the walls depends on (a global seismic response is possible only if vertical and horizontal elements are properly connected); c) the local flexural behaviour of the floors and the walls out-of-plane response are usually not modelled because they are considered negligible with respect to the global building response which is governed by their in-plane behaviour. As regard issue c), it has to be highlighted that “local” out-of-plane mechanisms may be verified separately through suitable analytical methods; moreover, in some softwares, the contribution of the out-of-plane stiffness is taken into account.

Within this general context, two main issues have to be solved, in particular related to: i) the strategy for assembling 2D masonry walls (discussed in the following); ii) the modelling of floors.

With reference to the first issue (i), different solutions may be adopted.

In some cases, the masonry portions defined as rigid nodes are modelled by means of rigid beams connected to the non-linear beams aimed to simulated piers and spandrel elements; in this case, usually the 3D model assembling is simply obtained by connecting these rigid beams, which work in separated planes, to the structural nodes at the insertion of masonry walls: as an example this solution is adopted in SAM II program (originally developed by Magenes & Della Fontana 1998, Magenes 2000). Differently, in case of TREMURI program (Lagomarsino *et al*. 2012), rigid end offsets are used to transfer static and kinematic variables between element ends and nodes: 3D nodes connecting different walls in corners and/or intersections are obtained by assembling 2D rigid nodes (characterized by 3 d.o.f.:) acting in each wall plane and projecting the local degrees of freedom along global axes (Galasco *et al*. 2004, Lagomarsino et al. 2013). Thus, in this case, the assembly is obtained by condensing the d.o.f of two 2-dimensional nodes by assuming the full coupling among the connected walls (Figure 4.85): the resultant 3D nodes have 5 d.o.f. in the global coordinate system, since the rotational d.o.f. around vertical Z axis is neglected because of the membrane behaviour adopted for walls and floors. This latter solution allows to reduce the total number of d.o.f. and to avoid potential numerical instability which could be associated to the introduction of rigid beams. It is worth noting that, in case of TREMURI program, having the 2D nodes no d.o.f. along the direction orthogonal to the wall plane, in order to guarantee the mass conservation along the 3 principal directions, the nodal mass

DRAFT

component related to the out-of-plane d.o.f. have to be properly shared to the corresponding ones of the two nearest 3D nodes of the same wall and floor (Galasco *et al*. 2004, Lagomarsino et al. 2013).

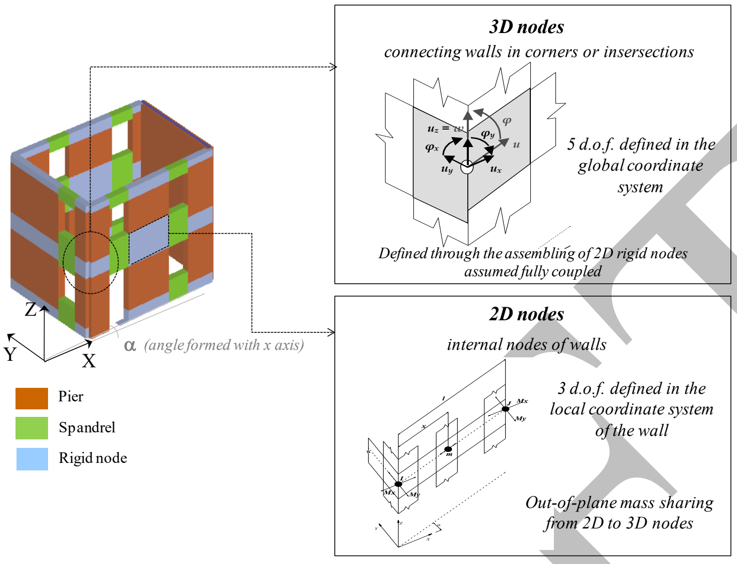

3D nodes
connecting walls in corners or insersections
$u_z = w$
φ_x
φ_y
φ
u
u_y
u_x
5 d.o.f. defined in the global coordinate system
Defined through the assembling of 2D rigid nodes assumed fully coupled
2D nodes
internal nodes of walls
3 d.o.f. defined in the local coordinate system of the wall
Out-of-plane mass sharing from 2D to 3D nodes
Z
Y
X
α (angle formed with x axis)
Pier
Spandrel
Rigid node

Figure 4.85. 3D assembling of masonry walls adopted in TREMURI program: classification of 3D and 2D rigid nodes

3.4.1.4 Modelling of diaphragms

With reference to the floors modelling, it is worth noting that a proper assumption on the diaphragm stiffness may significantly affects the overall response: in fact, in the limit case of “infinitely” flexible floors, there would be no load transfer from collapsed walls to still efficient structural elements. On the contrary, in the limit case of “infinitely” stiff floors, this contribution could be overestimated.

To clarify this issue, in the following the response of a very elementary structure (made up by a single cell) is examined (see also Cattari and Lagomarsino 2012). In particular, the walls quoted as P1 and P3, which the floor weights on, are characterized by a different stiffness (where P1 is more flexible than P3). Moreover, the masses associated to each wall are assumed to be equal and the piers as fully coupled (as a consequence the coupling effectiveness of spandrels does not affect the response). The results carried out by performing a pushover analysis with uniform load pattern are summarized in Figure 4.86, with reference to the case in which floor is assumed as infinitely flexible (a) or rigid (b), respectively. In particular, in case of the base shear (V) - displacement (u) curves, the following issues have to be stressed: the overall base shear V_{tot} is plotted with respect to the average displacement between P1 and P3 (u_{av}); the base shear associated to P1 and P3 (V_{P1} and V_{P3}) is plotted with respect to the corresponding wall displacement. Moreover, the shear amount shared through the floor progressing the non-linear response (F_{rip}=(V_{P3}-V_{P1})/2), normalized to the limit value $F_{rip,lim}$ is plotted as a function of P1 displacement (u_{P1}). In particular, with reference to a generic point **1** of the analysis (on V-u curve, which corresponds to **1’** and **1’’** points on V_{P1}-u_{P1} and V_{P1}-u_{P1} curves, respectively) it can be observed that: in case a), progressing the non-linear response, since no load transfer is possible ($F_{rip}/F_{rip,lim}$=0), when P1 reaches its maximum strength, P3 cannot further evolve (even it still stays in elastic phase); in case b), due to the coupling provided by floor, the displacements associated to P1 and P3 should

DRAFT

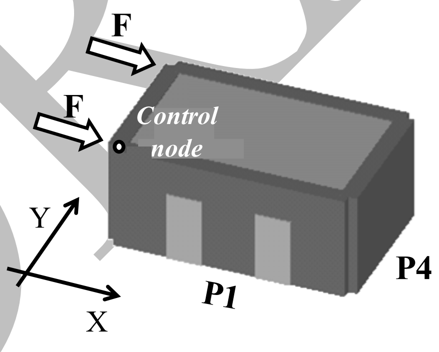
be equal (the slight difference is due to some torque effect), the wall P3 may attain to its maximum resources and the load transfer due to floor may asymptotically increase until its limit value. Despite the simplicity of the examined case, it is evident how the hypotheses assumed for floors may significantly affect the seismic response.

However, despite the evidence of this crucial role, the floor behaviour in 3D modelling is often assumed as rigid. This hypothesis may result quite rough and not acceptable in case of existing buildings and even more in case of monumental ones, where constructive technologies, like as timber floors or structural brick or stone vaults, may be found.

Actually, it is worth noting that in case of floor assumed as infinitely rigid, since it is sufficient apply the kinematic relationships of rigid body, its actual modelling isn't requested. On the contrary, in case of floor elements, modelled as orthotropic membrane finite elements (such as the solution adopted in TREMURI program) all the stiffness properties have to be defined.

In particular, in TREMURI program, floor elements, are identified by a principal direction, with Young modulus E_1, while E_2 is the Young modulus along the perpendicular direction, ν is the Poisson ratio and $G_{1,2}$ the shear modulus. The moduli of elasticity E_1 and E_2 represent the normal stiffness of the membrane: each one of them structurally describes the connection degree between the floor and the vertical wall parallel to its reference direction, both in linear and non-linear phases. The normal stiffness of the floor along X-axis provide a link between the piers of a wall parallel to X-axis, influencing the axial force the spandrels. Moreover, the $G_{x,y}$ parameter represents the shear stiffness of the floor and influences the horizontal force transferred among the walls, both in linear and non-linear phases.

DRAFT



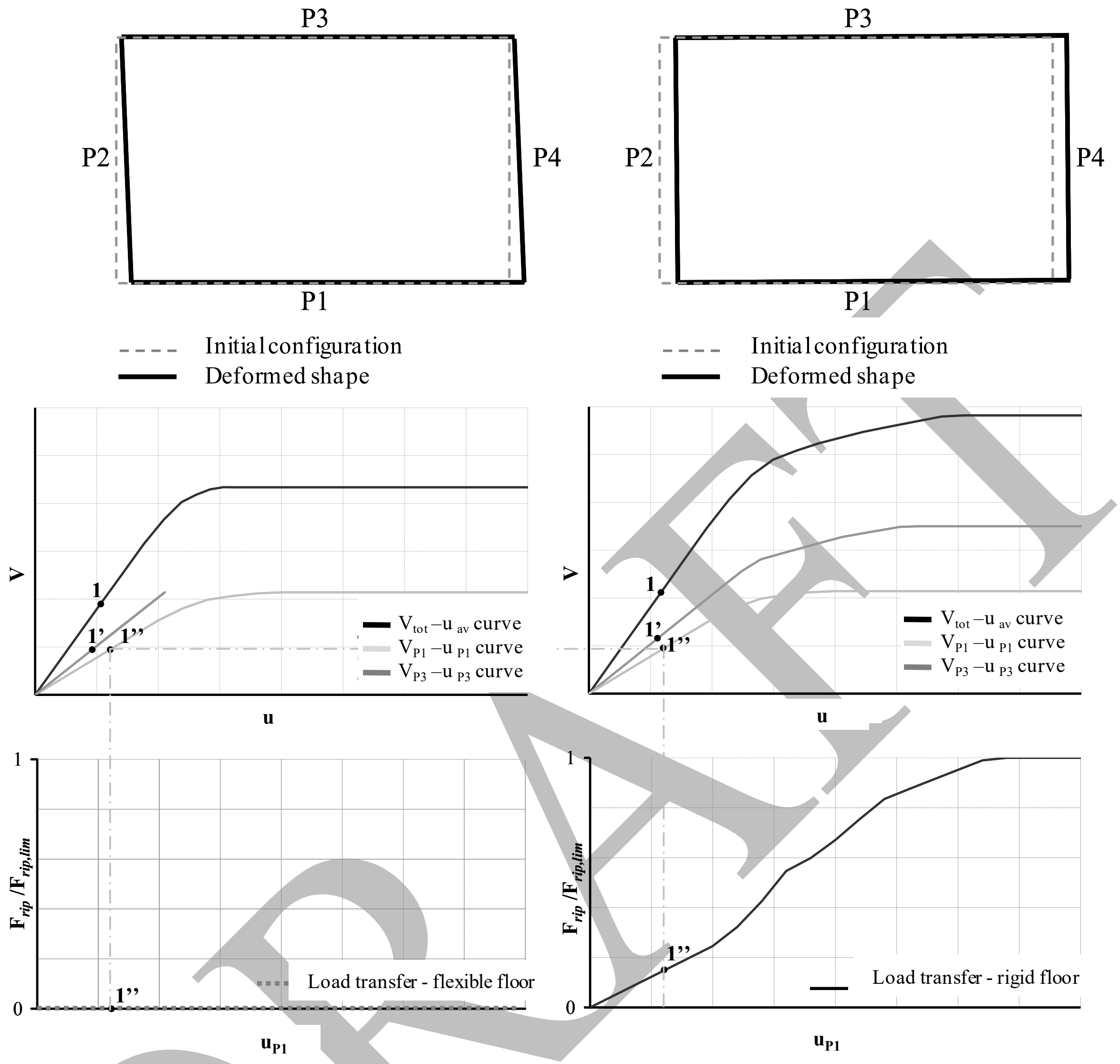


Figure 4.86. Load transfer effect as a function of the floor stiffness: infinitely flexible case (left); infinitely rigid case (right) (from Cattari and Lagomarsino 2012)

The evaluation of the abovementioned quantities may be rather simple in case of some floor typologies, ascribing it to the structural role shown by some specific elements. For example, in the case of a r.c. floor with beams and slab the shear stiffness is mainly given by the slab whereas the beam axial stiffness leads to the definition of E_1. On the contrary, in case of various ancient floor technologies, as the case of vaults, beside thickness and material properties, the stiffening contribution strongly depends on shape and geometrical proportion (e.g., rise-to-span ratio). Therefore, the definition of the elastic moduli to be attributed to the equivalent membrane element may be highly arbitrary. In Cattari et al. (2008), starting from the results of both linear and nonlinear FEM numerical simulations, the definition of equivalent stiffness properties has been proposed for some types of vaults (barrel, cross and cloister vaults) as a function of different thickness-to-span and rise-to-span ratios, constraints conditions and masonry texture pattern (parallel, orthogonal and oblique).

DRAFT

In case of wooden floors and roofs, some recent experimental works suggest a critical review of the formulae proposed in the literature and norms (Tomasi et al. 2009, Brignola et al. 2009, NZS 3603, ASCE/SEI 41-06).

Figure 4.95 shows (with reference to the basic example afore discussed) the different shear amount shared by floor as a function of structural type (and the corresponding stiffness); in case of vault types the expressions proposed in Cattari et al. 2008 to define the equivalent stiffness properties of membrane have been assumed as reference.

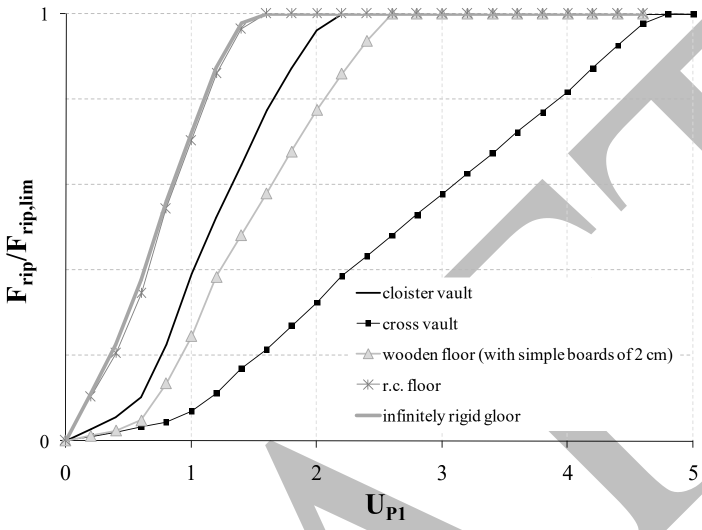


Figure 4.87. Load transfer effect as a function of the floor type (and their corresponding stiffness properties)

A further improvement of the formulation adopted for floors could be associated to the assumption of a bilinear response (such as elasto-plastic) also for these elements. Of course in this case, proper values of drift limits and ultimate strength values should be defined in order to establish the limits beyond which the floor loses the capability of transferring loads.

3.4.1.5 Modelling of foundations and soil

Usually pushover analyses are performed assuming a fixed-base structure. However, in some cases (e.g. for heavy and rigid masonry structures), compliance and geometry of the foundation system, in combination with the non-linear behaviour of the foundation soil, may significantly modify the actual response in terms of both capacity and demand. In order to improve the reliability of seismic assessment in case of Soil-Foundation-Structure Interaction (SFSI), impedance functions for flexible masonry foundations (calibrated on basis of detailed numerical analyses) are proposed, in order to be easily implemented in engineering practice.

The soil-foundation system may be modelled:

- by means of proper macroelements aimed to take into account in a more accurate way also the actual non-linear response of such system;
- by means of springs (Beam on nonlinear Winkler foundation model) for which the stiffness is calibrated in a proper way - on basis of proper impedance functions - to take into account for different conditions (rigid or flexible foundations). In this case, the foundation system may be

DRAFT

included in the model (by a proper mesh defined on basis of its geometry) or assigned directly to the base restraints of structure (without the actual modeling of foundation system).

3.4.2 Modelling by macro-block models (MBM)

The observation of the main damage patterns in post-earthquake surveys highlights the propensity of historical masonry structures to develop damage modes involving the formation of macro-blocks, which collapse because of the loss of equilibrium rather than the reaching of an ultimate resistance of the masonry material. Furthermore, the damage occurred after past earthquakes showed that the damage of elements mainly occurs in just some concentrated zones (while the other parts of the elements remain undamaged) and the structure is divided into different blocks, which can be assumed as rigid (Figure 4.88).

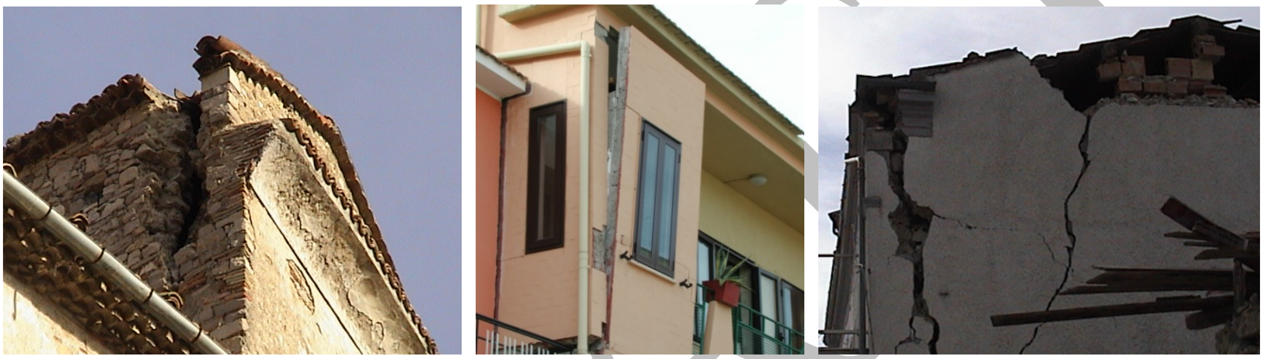


Figure 4.88. Examples of occurred mechanisms involving macro-blocks

For these reasons, limit analysis tools can be legitimately used.

The Limit Analysis is an effective analytical tool to define the bearing capacity of a structure, by calculating the value of the loads multiplier▯▯$_{▯}$▯that corresponds to the coming of the system to an equilibrium limit condition. Two different approaches can be used: the static approach ("incremental analysis") and the kinematic one ("analysis for kinematisms"). Usually, the latter approach is mainly used; however, the validity of both approaches is established by the following main assumptions (Heyman, 1966, 1982, 1995): rigid masonry blocks, infinite compressive strength, no tensile strength and no sliding occurrence.

In general, discrete model approaches (like those based on the use of macro-block models) can be used to analyse local mechanisms, generally associated to different failure modes that may occur in masonry historical buildings and assets involving portions of them. In case of local mechanisms, reference is made to the response of single parts of the structure and in most of cases, these parts are "macroelements". The use of "macroelement" word refers to portions of an architectonic asset for which, as testified by the earthquake damage survey, it is possible to recognize recurring and autonomous seismic behaviour. A macroelement may include a set of structural elements (as in the case of a wall, in which piers and spandrels are included) or, in some cases, may coincide with the structure itself (as in the case of a tower).

More in detail, a local mechanism is defined by a set of masonry blocks (assumed as rigid), connected one each other through internal constraints (such as hinges) and element of connections (rigid, elastic or which are controlled by friction actions), which are able to simulate the presence of steel tie rods, beams or connection between masonry panels; furthermore, some external constraints are present, able to simulate

DRAFT

DRAFT

the connection of the portion interested by the mechanism with the rest of the construction. The whole set of these constraints determine a single degree of freedom kinematic chain, whose motion act is described by a displacement (or rotation) variable.

In order to study the examined mechanism seismic response, it is necessary preliminary to individuate the value and the application point of all the loads (own weights and loads applied to each block), of all the external forces applied to each block and of all the internal forces in the elastic or friction elements; then, a system of horizontal forces proportional through a coefficient α to the masses, that are activated by the seismic action, is considered.

It is important to point out that it is necessary to take into account also the possible presence of loads which are not directly applied to the blocks, but whose seismic inertial action would weight on the blocks of the mechanism (for instance, a floor or a roof weight on the kinematism just with a part of their weight, but if not connected, determines a horizontal seismic action proportional to the entire mass).

Finally, by applying the Virtual Works Principle to the infinitesimal motion act, it is possible to calculate the value of α_0 that activates the kinematism as follows:

$$\alpha_0 = \frac{\sum_{k=1}^{N} P_k \delta_{Py,k} - \sum_{k=1}^{m} F_k \delta_{F,k} + L_i}{\sum_{k=1}^{N} (P_k + Q_k) \delta_{PQx,k}}$$

where:

- N is the number of blocks which characterizes the kinematic chain;
- m is the number of external forces, assumed independent from the seismic action, applied to the different blocks;
- P_k is the resulting of the weights-forces applied to the *k*-th block (block own weight, applied in its center of gravity, plus the other applied loads)
- Q_k is the resulting of the weight-forces not directly applied to the *k*-th block, but whose mass determines on it a horizontal seismic action, due to the fact that it is not efficaciously transmitted to another part of the structure;
- F_k is the generic external force applied to one of the blocks;
- $\delta_{Py,k}$ is the virtual vertical displacement of the centre of gravity of the whole weight-forces P_k, acting on the h-th block, assumed as positive if upwards directed;
- $\delta_{F,k}$ is the virtual displacement of the application point of the external force F_k, projected in its direction;
- $\delta_{PQx,k}$ is the virtual horizontal displacement of the centre of gravity of the horizontal forces $\alpha(P_k+Q_k)$ acting on the *k*-th block, assuming as positive the direction of the seismic action which activates the mechanism;

- *L_i* is the total work of the possible internal forces (tie rod extension, sliding with friction in presence of good connection).

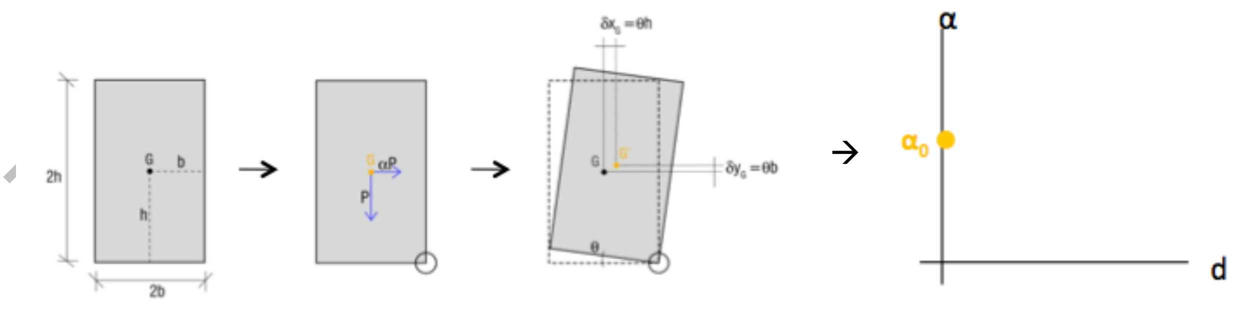
The above mentioned equation to calculate the value of ⍰$_0$ that activates the kinematism and corresponds to the application of the Limit Analysis through the linear kinematic approach (Figure 4.89); the obtained multiplier ⍰⍰ estimates in a not precautionary way the actual static collapse multiplier.

If the part of construction identified by the system of blocks is assumed as infinitively rigid until the kinematism activation, the multiplier ⍰⍰ represents the peak acceleration (in *g* units) of the structure motion at the level where the mechanism is connected with the rest of the structure (if the examined system is a single block connected to the ground, ⍰⍰ represents the peak ground acceleration).

In order to evaluate the displacement capacity of the local mechanism until the collapse, it is possible to evaluate the horizontal multiplier ⍰ not only in its initial configuration, but also in the consecutive varied ones, which represent the development of the examined kinematism and which are described by the horizontal displacement d_c of a control point of the system, anyway chosen. If no internal resistance forces are present (which can increase with the displacement), the multiplier gradually decreases until the configuration which corresponds to ⍰=0 and d_c=d_{c0}; physically, where the block loses its equilibrium in a static condition (Figure 4.89).

DRAFT

a) LINEAR KINEMATIC ANALYSIS TO DETERMINE ⍰$_0$



b) NONLINEAR KINEMATIC ANALYSIS TO DETERMINE ⍰-d$_c$

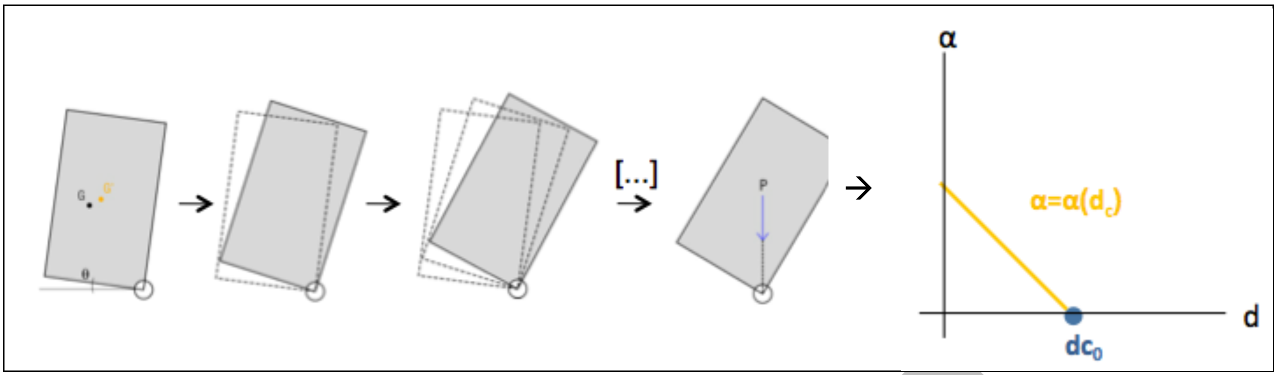


Figure 4.89. Scheme of the procedure of analysis for a single rigid block

The curve ⍰-d_c, obtained through the nonlinear kinematic analysis, is the pushover curve of the local mechanism; in order to determine it, it is necessary to consider how the internal and external forces change and if they persist during the kinematism development.

For instance, in a steel tie rod subjected to an initial tensile load, the load increases until the steel yield stress, then it remains approximately constant and finally it determines the break of the tie rod; consequentially, the steel tie rod is no more able to contribute, and the curve comes back to that one representative of an analogous block without tie-rod (Figure 4.90).

A good connection between the different masonry walls contributes thanks to the friction forces until the detachment between masonry blocks, while a floor transfers the forces until its beams are pulled out from masonry (if it is considered that the pulling out of the floor beams determines the collapse of the structure, the pushover curve has to be stopped further than this condition happens).

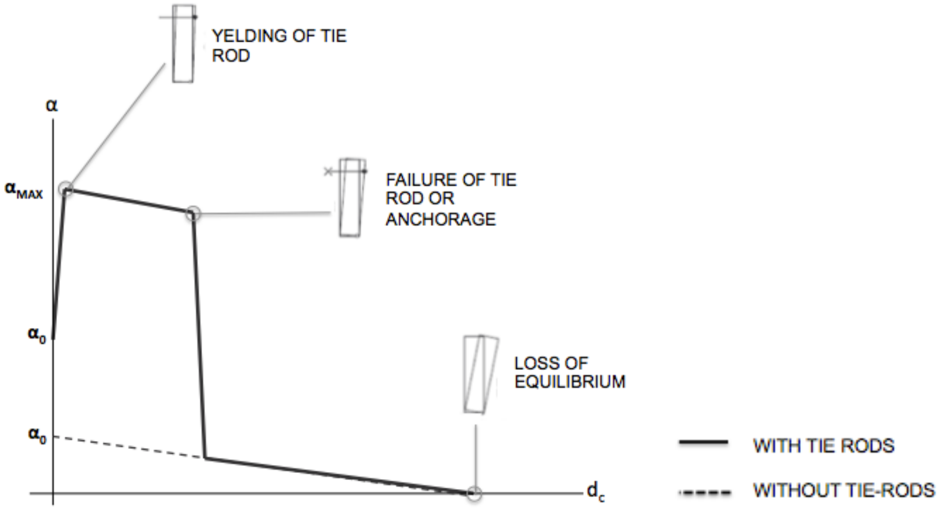


Figure 4.90. ⍰-d_c curve obtained by a nonlinear kinematic analysis with (continuous line) and without (dotted line) tie-rods

Further information and suggestion are provided in following sections concerning the identification of collapse mechanisms, the influence of masonry quality and of connections between masonry walls (masonry pattern, presence of steel tie rods) and the interactions with other construction elements or with adjacent buildings.

DRAFT

3.4.2.1 Identification of the collapse mechanism

The use of the kinematical approach presumes substantially to analyse all local mechanisms which are considered significant and possible for the examined structure, according to a prediction made through the presence of light cracks in the building or from the comparison with similar structures already damaged. The behaviour of a discrete set of masonry bodies, connected through interfaces, is then considered; the shape of each body is defined on the basis of recurrent crack patterns observed in post-earthquake surveys (each masonry body is commonly assumed as rigid) and by taking into account the presence of seismic or pre-seismic damages, which might have already occurred.

The identification of structural masonry portions to be modelled through discrete models, it's interesting to highlight that for MBM the shape of each body is defined "a priori" on the basis of recurrent crack patterns observed in post-earthquake surveys. For this reason, the reliability of the analysis is firstly connected to the correct choice of the collapse mechanism; consequentially, the individuation of the correct mechanism presupposes an adequate knowledge of the constructive details, a careful evaluation of the efficaciousness of the elements of connection and a correct interpretation of the occurred crack pattern.

3.4.2.2 Modelling of limited compressive strength

As above mentioned, the validity limits of MBM are mainly related to the simplified hypothesis adopted, which basically can be summarize as: infinite compressive strength, no tensile strength and no sliding occurrence (Heyman, 1966, 1982, 1995). Then, when the assumptions made result too far from the reality, other more refined modelling strategies should be adopted. These alternative strategies concern mainly the modelling of limited compressive strength and of connections.

Regarding the *modelling of limited compressive strength*, the assumption of considering infinite masonry compressive strength is correct when masonry walls with good characteristics (regular patterns and transversal connection between the leaves) is considered; in this case, local collapse mechanisms develop as loss of equilibrium of masonry portions with a monolithic behaviour, which may be easily analysed with rigid macro-block models. On the contrary, for poor masonry made up of irregular stones or multi-leaf masonry with poor connection between leaves, the hypothesis of monolithic behaviour is not expected to be valid; in this case, it could be more appropriate to adopt DIM models or a refined MBM approach. For example, as recently demonstrated by experimental and numerical research campaigns (De Felice 2010), the out-of-plane seismic capacity of irregular stone masonry walls with limited compressive strength may be modelled by the nonlinear kinematic analysis of a rigid block whose base hinge (centre of rotation) is shifted inside the section (Figure 4.91).

DRAFT

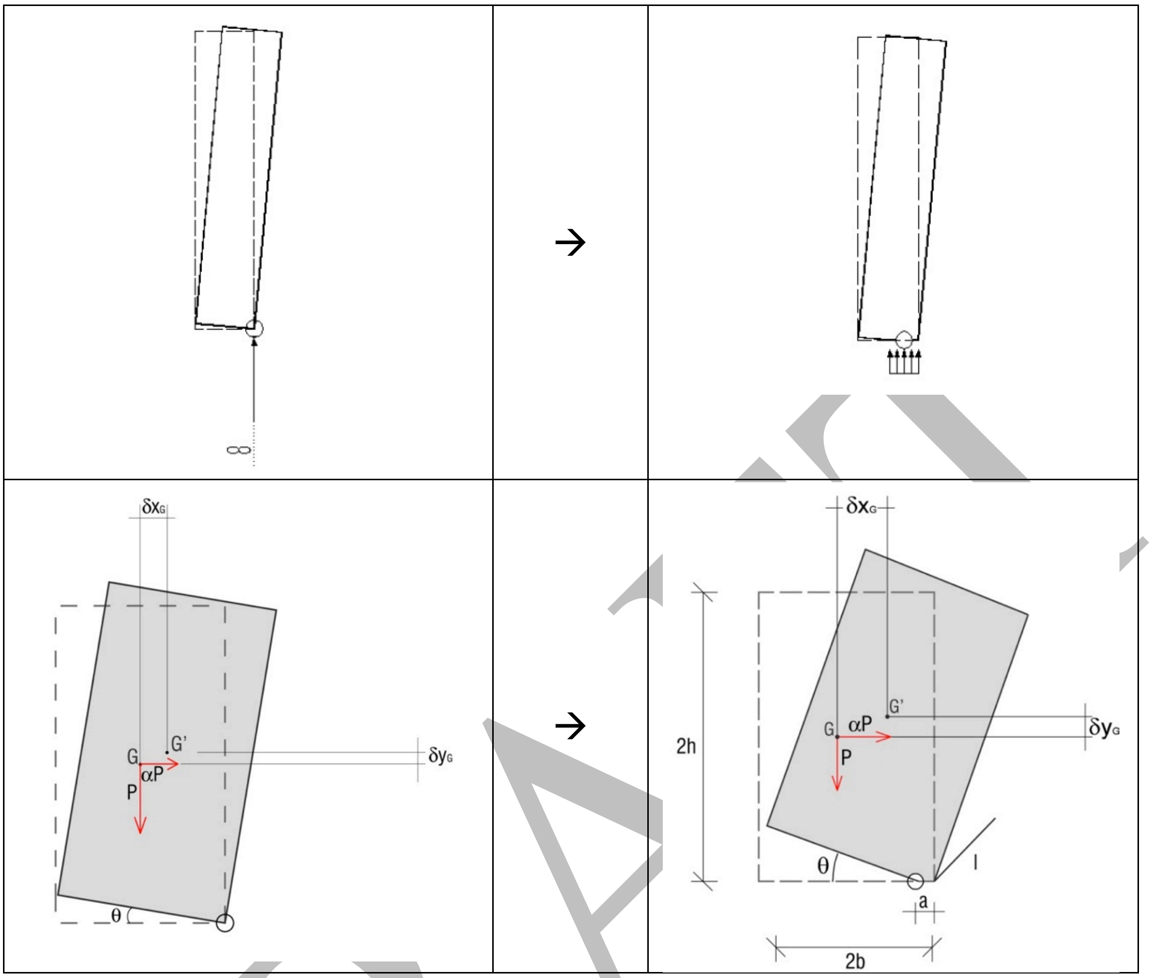


Figure 4.91. Modelling of masonry limited compressive strength

The value of the multiplier $\alpha=f(\theta)$ - where θ is the angle which describes the block overturning - is consequentially determined as:

$$\alpha = \frac{1-\bar{\theta}\lambda-\beta\lambda}{\lambda+\bar{\theta}(1-\beta\lambda)},$$

where $\lambda=2b\gamma/f_m$ is a coefficient which depends from the geometrical dimensions of the considered block (b), the density γ and the value of the compressive strength of masonry f_m.

As a consequence, the value of α_0 which activates the kinematism is:

$$\alpha_0 = \frac{1-\beta\lambda}{\lambda}$$

By taking into account the limited compressive strength, the value of α_0 and $d_{c,0}$ is a bit lower than under the assumption of unlimited resistance (Figure 4.92).

DRAFT

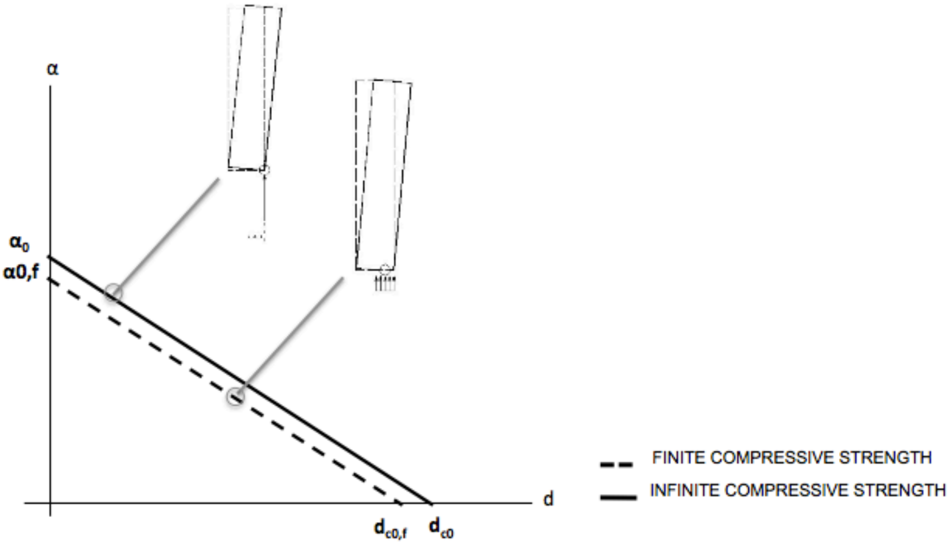


Figure 4.92. Comparison with the results in terms of curve �-d, considered finite or infinite the compressive strength of masonry

3.4.2.3 Modelling of connections

Regarding the correct modelling of the connections it is important to preliminary evaluate:

- the presence and the real effectiveness (such as tie rods, curbs or other connections between floors and walls);
- the presence of thrusting elements, not adequately contrasted (vaults, arches, rafters);
- the quality of the interlocking: between orthogonal walls (corners or internal walls); between floor and wall (in this case, it is important to evaluate also the limit condition of pulling out, during the mechanism progressing); the presence of propping elements (such as buttress or adjacent structures).

In particular, regarding the role of the connections, it is possible to distinguish substantially two different cases:

- completely effective connection between the overturning block and the orthogonal walls;
- breaking of the connection between the overturning wall and the orthogonal walls.

Completely effective connection

In this case, in general the mechanism involves the overturning of the element together with a detached portion of the walls connected (which can be perimeter or internal); the detached masonry block is evaluated in relation to the masonry texture, by supposing an opportune inclination ϕ. In general, the angle ϕ (formed by the diagonal of the masonry block and the vertical axis) increases with the ratio between the horizontal

DRAFT

DRAFT

and vertical size of the masonry units (which is regular in case of brick or dressed stone masonry, while for cut stone masonry a representative value may be assumed). Usually, as precautionary evaluation, it seems legitimate assumed the lower values of ⍰=15° (in the case of masonry made by rubble stones or uncut stone and poor mortar) and higher values till to ⍰=30° (in the case of brick masonry or dressed stone masonry). Furthermore, the geometry of the masonry blocks can change as a function of the presence of openings. Effective connections are in general present in masonry structures built in one construction phase or that were in the past subjected to strengthening interventions to the corner parts (where the connection is necessary), for example through the introduction of steel tie-rods.

Figure 4.93 illustrates an example of MBM model that takes into account the presence of a good connection between the façade and the internal walls (blocks named A and B); the masonry block B geometry is different because of the presence of some openings in the internal wall.

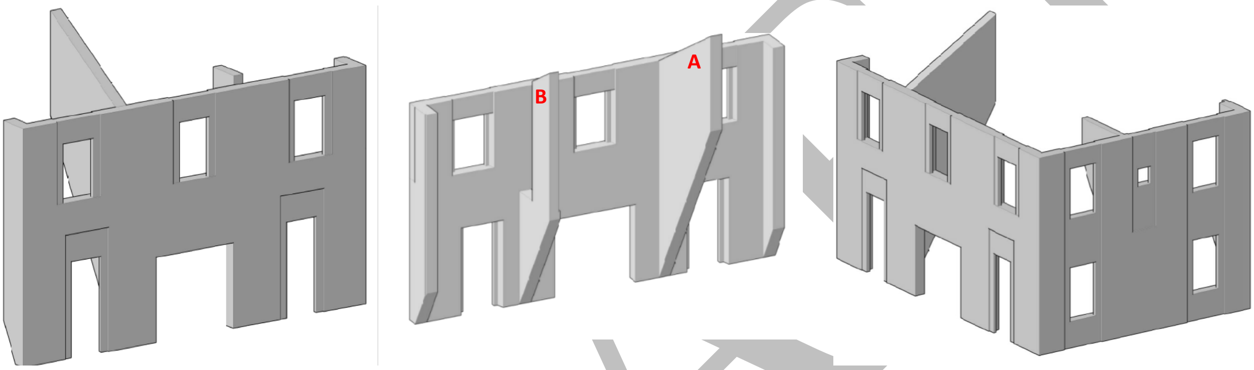


Figure 4.93. MBM models that take into account the presence of a good connection between the walls

For instance, in case of the masonry block B (made by solid brick masonry) it is possible to evaluate the value of the resistance force per unit of length that the good connection is able to develop, as follows:

$$f(z) = \frac{\tau(z) A_{amm}}{2b}$$

where :

- $\tau(z) = \mu\left(\gamma z + \frac{Q}{s}\right)$ shear stress which is developed due to the friction effect on the interlocked surface; it is proportional to the compression acting on the contact surface;
- $A_{amm} = s \cdot \frac{a}{2}$ surface of superposition between masonry units (a is the horizontal size of masonry units);
- b height of the masonry unit plus thickness of the mortar joint;
- µ Friction coefficient;

- γ unit weight of masonry;
- z Distance of the position of the contact surface from the upper part of the macro-element

Once the *f*(*z*) has been determined, it is possible to calculate the resulting force *F* and its position (Figure 4.94). In the examined example:

$$F = \frac{\left(f(0) + f(H)\right)H}{2}$$

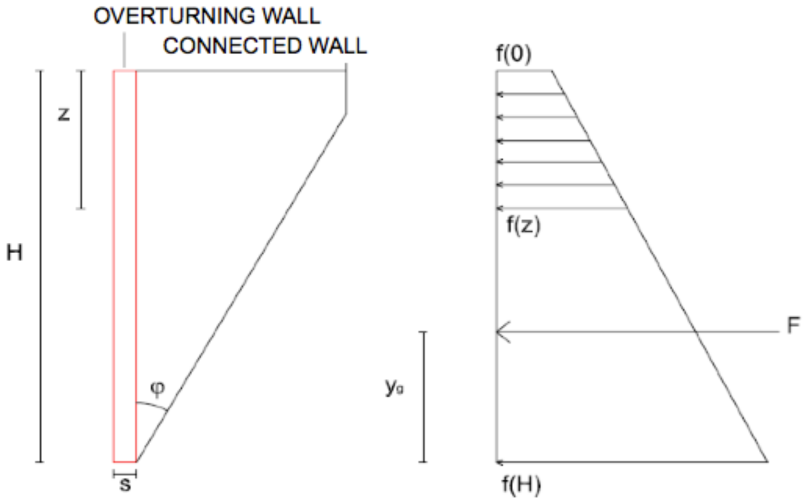


Figure 4.94. Distribution of the resistance force per unit of length due to the good connection between facade and internal wall

Fracture of the connection

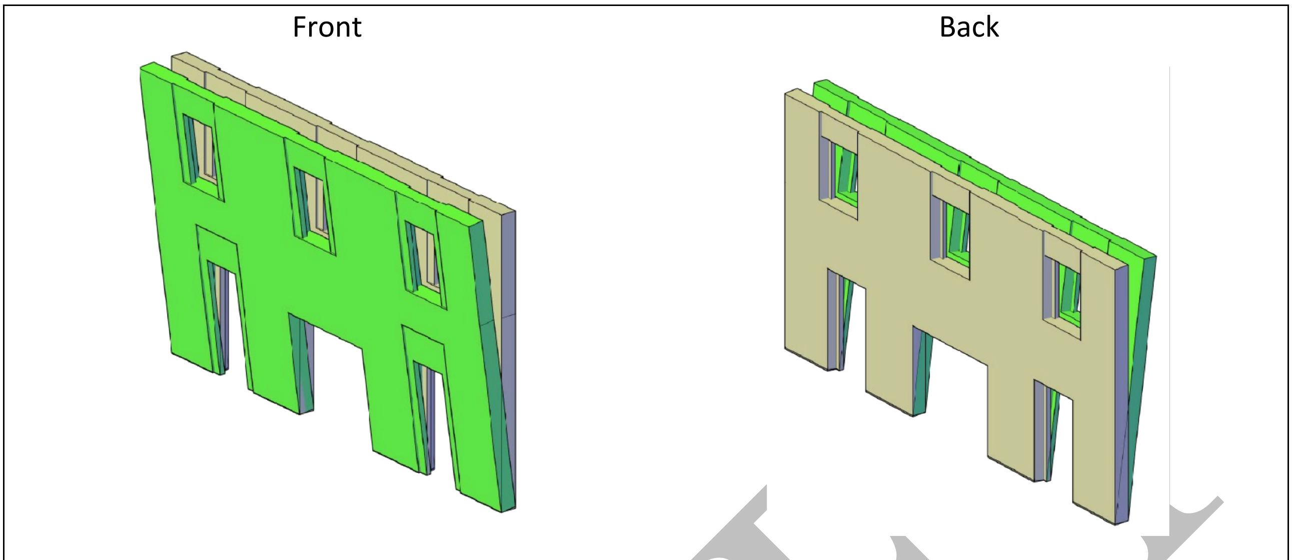
In this second condition, it is possible to distinguish between two different cases:

- the connection is completely ineffective, so that it is completely ignored in the model;
- the fracture involves head joints but there is a contribution offered by the shear friction due to sliding in the bed joints (until a complete detachment); in this case no portion of the orthogonal walls is included in the mechanisms.

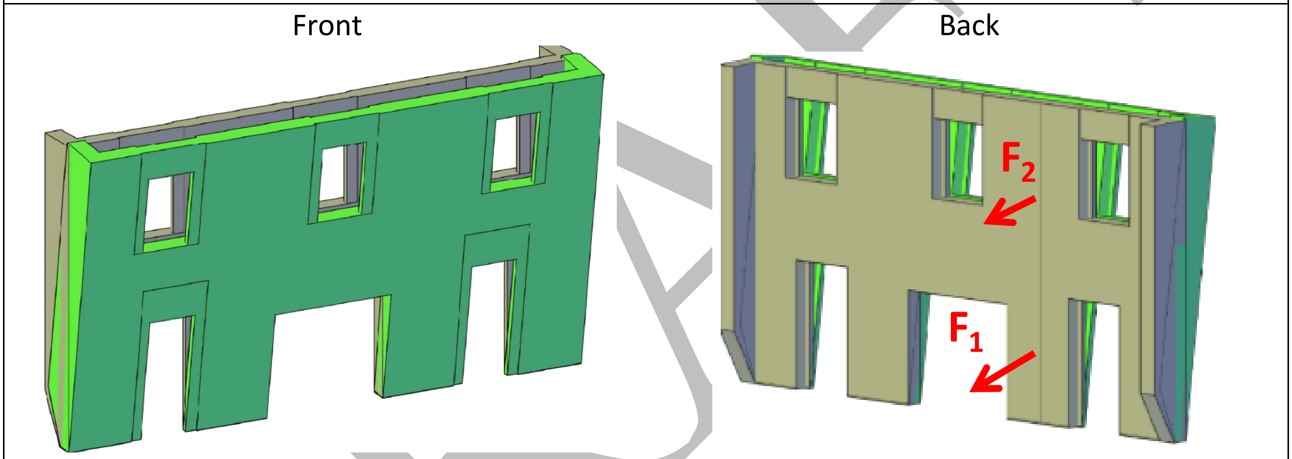
Figure 4.95 illustrates two examples of these cases: in the first one (above), the connection between the façade and the internal walls are completely ineffective, so that it is not modelled at all; in the second one (below), the connection is considered as a horizontal force F (effective just to the breaking of the connection), without modelling the masonry blocks corresponding to the connected parts of the internal walls, in order to ignore its stabilising contribution.

DRAFT

DRAFT



CASE 1: COMPLETELY INEFFECTIVE CONNECTION



CASE 2: FRACTURE OF THE CONNECTION

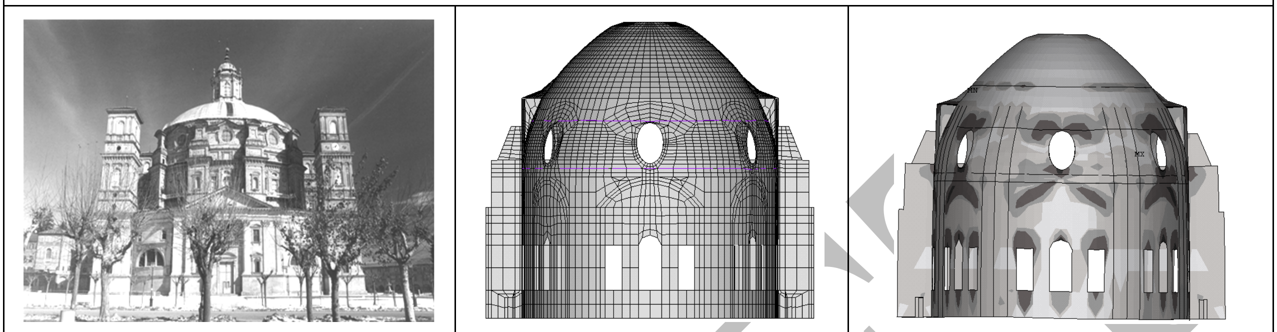
Figure 4.95. Different modelling strategies of the connection between facade and internal walls, depending on its effectiveness.

3.4.3 Modelling by Continuum Constitutive Laws Models (CCLM)

3.4.3.1 Choice of the elements and mesh size

Nonlinear Continuum Constitutive Laws are commonly adopted in conjunction with two or three-dimensional Finite Element Models, which allow to perform both incremental and collapse analyses. The material is considered as homogeneous and the structure is thus described by a continuum mesh. Modelling through finite element require a choice regarding material discretization and, in particular dimension of finite elements. It is well evident that the discretization could be nor too fine, due to computational problems nor too quite coarse. As general rule, it can be said that a ratio 1/3 between the dimension of finite element and the linear dimension of structural elements (such as piers) is the lower limit to be able to get information on shear and bending moments, while a ratio 1/10 ÷ 1/20 is the upper limit to let the model computationally acceptable. Obviously, the choice depends on the scale of the model and on aims. If a complete or near complete model of the structure should be made, then a coarse discretization is unavoidable (Figure 4.96) whereas if a detailed model of single macroelement is required, a finer mesh could be adopted (Figure 4.96).

a. Complex masonry structure



b. Simple macroelement

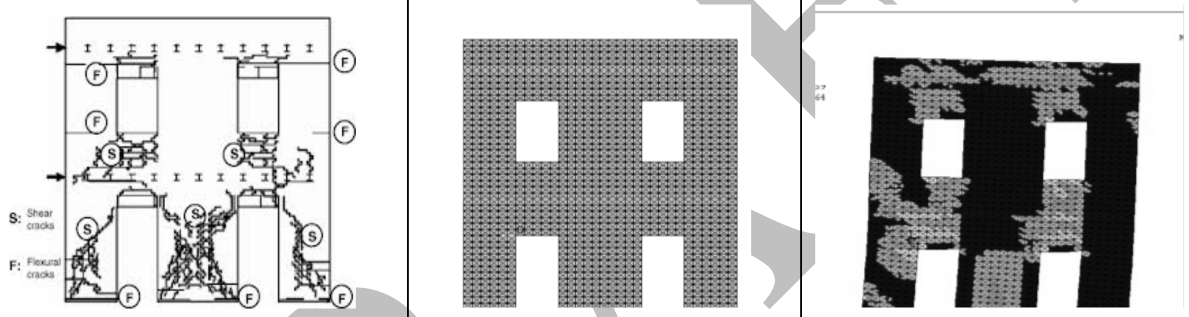
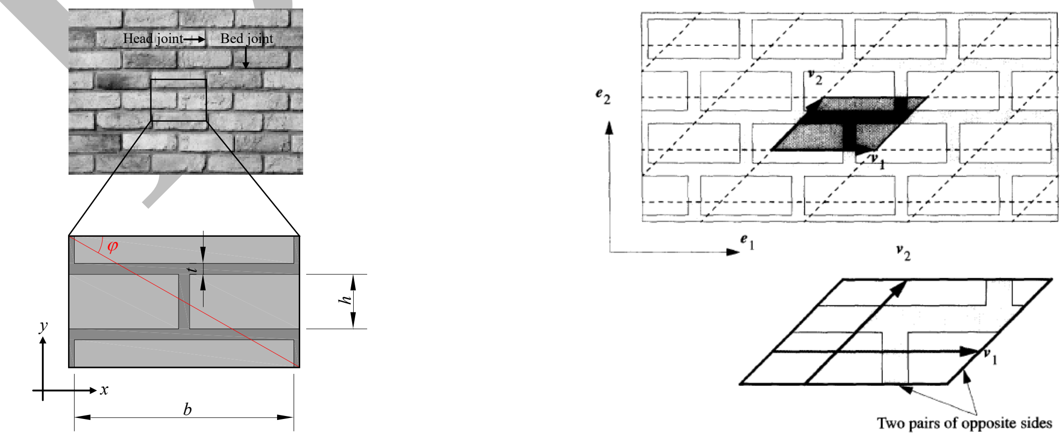


Figure 4.96. Modelling of the masonry structures with FEM and CCLM: (a) large scale modelling with coarse discretization and (b) macroelement scale modelling with fine discretization.

It is worth noting that, by adopting certain types of nonlinear constitutive laws based on homogenization, the mesh could not too fine due to theoretical incompatibility between the size of the reference volume for homogenization and the size of finite elements. Two examples of typical reference volumes are in Figure 4.97: in both cases, the size of the finite element should be not smaller than $b \times 2h$, b being the width of the brick and h it height.

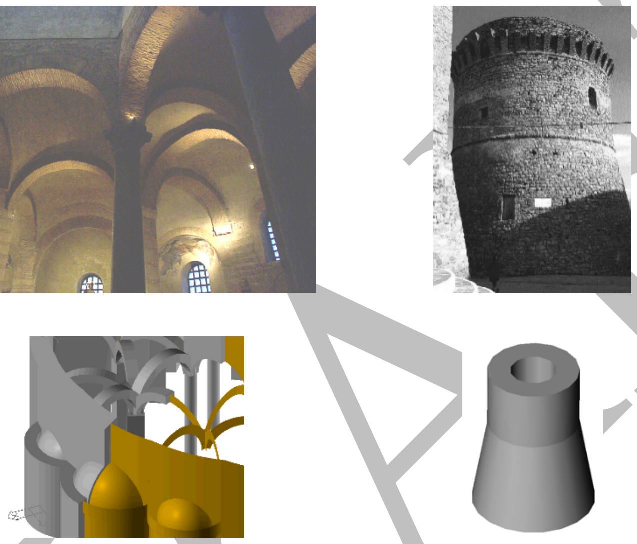


DRAFT

a. Calderini and Lagomarsino (2008) b. Anthoine et al. (1995)

Figure 4.97. Example of reference volumes adopted for the homogenization of masonry: in both cases, the size of the finite element should be not smaller than *b*×2*h*.

In general, both two and three-dimensional finite elements may be adopted (the integration of solid and shell elements should be considered carefully). The choice depends on the type of masonry structure. In general, for the modelling of building of class A, B the adoption of two-dimensional shell element is effective since the thickness of their constituting walls is limited in relation to the other two dimensions. For building of other classes the choice of three-dimension finite element may be considered. It is the case of massive structures such as towers (class C), triumphal arches (class D), defensive city walls (class E).



a. b.

Figure 4.98. Modelling of the masonry structures with FEM and CCLM: church with thin walls (a) and tower with thick walls (b)

A particular care should give to the choice of shell elements, since they should be able to describe the on-of-plane behaviour of masonry. In many finite element codes, there are nonlinear shell elements based on a discretization through the thickness (3 to 5 integration points).

The use of shell element and of a bi-dimensional idealization of masonry walls requires the choice of the reference surfaces to be considered in the model. Usually, the mean plane is adopted (see Figure 4.98) but this is not the only possible choice. In many cases, problems may raise from the intersection of perpendicular walls or arches, implying the stretching of the geometry of structural elements in order to fit the nodes on the intersection.

3.4.3.2 Modelling of walls and vaults

In modelling masonry walls most of the problems comes from the presence of openings. As well known, openings in masonry structures may be realized in two ways: by lintels made of wood or stone or by arches.

DRAFT

In the first case, in the finite element model beam elements should be introduced. In general, such beam may be elastic, since lintels are usually quite resistant elements that under static conditions are subjected to low tensile stresses produced by the voids and under seismic actions are unlikely to fail before masonry due to bending or normal effects. However, just because they are resistant elements, it is important to avoid that they constitute, unintentionally, a chain for masonry spandrel. In fact, lintels are simply set on the masonry wall, without any connection. This implies that they extremities are not allowed to move along the vertical (gravity force) direction, but, may have differential displacements with masonry (whose entity depends on the friction of the contact surface) in the horizontal direction. For this reason, they do not work as chains in real buildings. In order to allow horizontal displacements of lintel extremities it is important to replicate the nodes on which lintels and masonry converge and rigidly couple the required degree of freedom (vertical motion). Another, more sophisticated possibility is to couple them with proper nonlinear link elements, able to describe the friction behaviour of the contact surface.

In the second case, in which arches should be modelled, it is very important to take into account the anisotropic behaviour of the material. This implies the identification, in each point of the masonry solid, of local axes of the material. In the model, such local axes may be easily attributed to finite elements by rotating their local reference system. An example is presented in Figure 4.99.

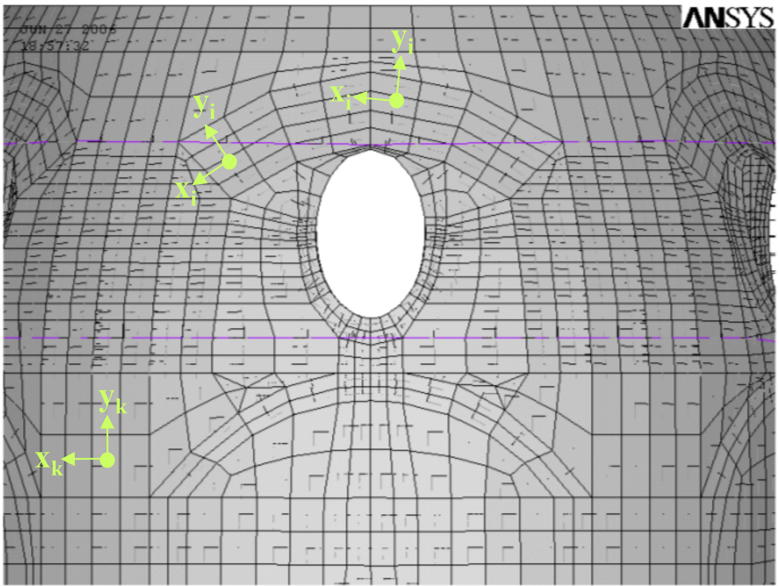


Figure 4.99. Modelling of arch openings in masonry structure: rotation of the local reference system of the elements to take into account masonry anisotropy.

The problem of taking into account of the local reference system of masonry is particularly evident in the modeling of masonry vaults or domes. In this case, in fact, the texture of the material should be necessarily considered in order to represent their actual structural response. Also in this case, the local reference system of the finite elements should be rotated in order to agree with the building one (Figure 4.100).

DRAFT

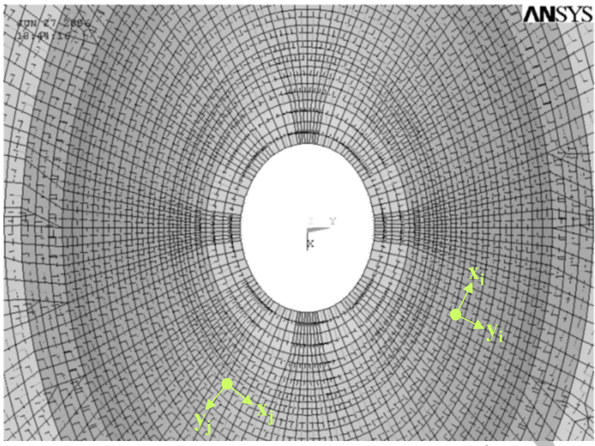


Figure 4.100. Modelling of dome: rotation of the local reference system of the elements to take into account masonry anisotropy.

In vaults and domes, some modeling problem raise from the curvature of such macroelements. Usually finite element discretization is based on plane elements. However, recently, some research works was specifically oriented implement nonlinear constitutive laws in curves elements. An example is the work of Milani et al. (2008), in which a new six-noded triangular curved element, specifically developed for the kinematic limit analysis of masonry shells, was presented. In this model, plastic dissipation is allowed only at the interfaces (generalized cylindrical hinges) between adjoining elements for combined membrane actions, bending moment, torsion and out-of-plane shear, as it is required for the analysis of thick (Reissner–Mindlin) shells. Masonry strength domain at each interface between contiguous triangular elements is evaluated resorting to a suitable upper bound FE homogenization procedure.

3.4.3.3 Modelling of diaphgrams

In finite element modelling of masonry structures diaphragms may be modelled in two different ways. The first choice is that of a detailed modelling approach, including thus the description of each single timber elements by beam-type elastic finite elements and, eventually, the description of the floor plane through elastic shell elements (Figure 4.101).

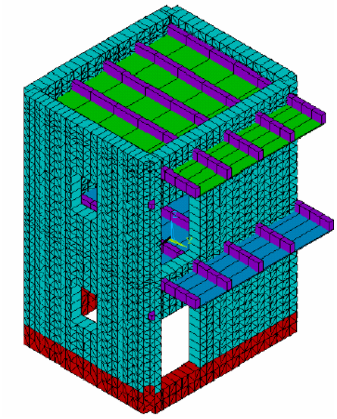


Figure 4.101. Modelling of floor through beam and shell elastic element.

DRAFT

Another choice, more simple, it that of modeling the diaphragm by a simple plane of shell elements, whose stiffness should be carefully calibrated.

3.4.3.4 Required geometrical and mechanical data

For the definition of finite element model, quite accurate geometrical data are required. Usually, the geometrical survey of the building is firstly transformed in a 3D geometrical model of the structure (by fitting the available data and, when irrelevant from the structural point of view, by simplifying the geometry of architectonic elements). Then, if the finite element model is based on 3D finite elements, the geometric model is kept as it is, imported in the FEM code and discretized. If the finite model is based on 2D finite elements, it is necessary to make a transformation reducing each solid masonry element in its representative surface; then, the new geometric model is imported and discretized.

The base parameters to be acquired for this type of models are the elastic modula of masonry. The other depend on the type of constitutive law adopted.

If the constitutive law adopted is based on a phenomenological approach, the parameters are overall properties of the composite that should be defined or on the basis of experimental tests performed on quite large structural elements (such as masonry wallets, piers etc…), or on literature works on similar cases. The parameter required are usually the compressive and the tensile strength of masonry, the shear strength of masonry. Moreover, in plasticity or fracture mechanics-based models, other less intuitive parameters are required, such as the fracture energy *G* or the softening ratio.

If constitutive laws based on micromechanical approaches are adopted, the parameters are usually referred to the single components of the masonry. In general, this highlight one of the advantage of such modelling approaches: the tests to obtain the mechanical parameters of single constituents (mortar and bricks) are surely less invasive than those to obtain overall parameters. In most of the models, besides the elastic modula, the compressive strength of bricks (or an equivalent compressive strength of small assemblages) is required, as well as the friction on mortar joints and the cohesion of mortar. Also in this case, in plasticity or fracture mechanics-based models, other less intuitive parameters are required, such as the fracture energy *G* or the softening ratio of mortar joints and bricks.

3.4.4 Modelling by Discrete Interface Models (DIM)

3.4.4.1 Main features and application

In discrete interface models, masonry is modelled as a discrete system of elements: blocks, joints and/or interfaces. Usually the nonlinearity of the material is concentrated in joint and interfaces. Many different approaches may be used.

In the approaches based on limit analysis, blocks are considered as rigid and infinitely resistant bodies interacting with nonlinear frictional joints. They widely used due to their low computational effort and are applicable to whole buildings.

DRAFT

In Distinct Element Method (**Errore. L'origine riferimento non è stata trovata.** - (Van Mele et al.,2012), blocks a re supposed to be rigid. For joints, a soft contact formulation (in which contact stresses are function of relative block displacements) is proposed. It may be used as well for static and dynamic analyses of masonry, through the explicit integration of the dynamic equations of motion. Its adoption to simulate dynamic response of structures is, of course, very suitable and allows for fully dynamic analysis with large displacements. However, its computational effort is quite high and, for this reason, its use is suggested for simple multi-block structures (life those of class F - Figure 4.102 - De Jong and Vibert, 2012) or to detailed models of single macroelements only (Figure 4.103).

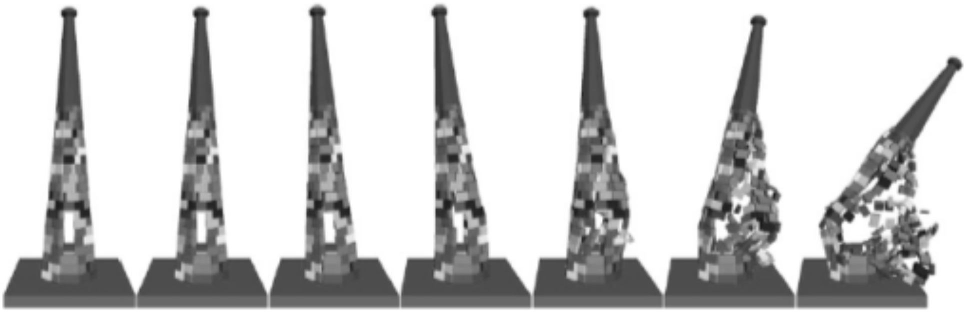


Figure 4.102. Example of a discrete element model of a spire and its progressive collapse due to pulse input (De Jong and Vibert (2012))

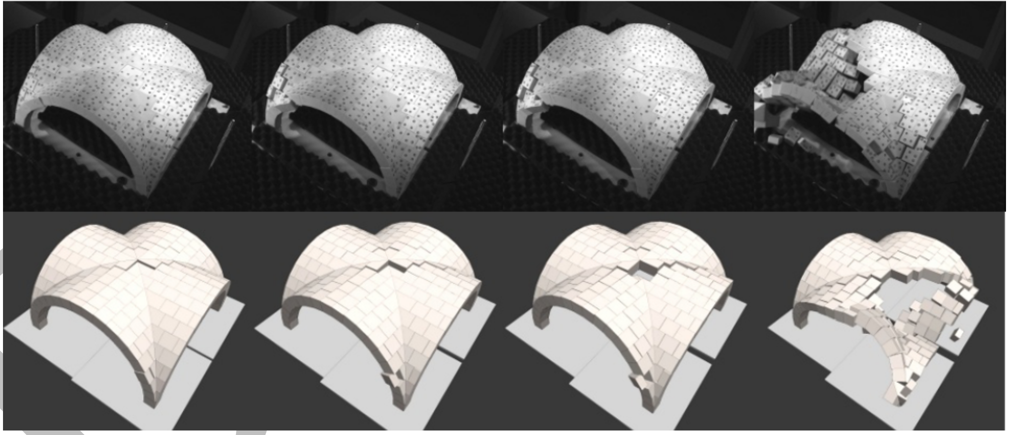


Figure 4.103. Comparison between experimental tests and DEM simulation on a cross vault (Van Mele et al. (2012))

Other possible modelling approaches are based on Discontinuous Deformation Analysis (DDA), in which blocks are considered deformable but in state of uniform stress and strain and contact are assumed to be rigid (no penetration between blocks is permitted), and on Finite Element method. The application of such technique involves substantial computational time; for this reason, do not find extensive applications in real practice.

DRAFT

3.4.4.2 Required geometrical and mechanical data

Generally, among the different typologies of DIM, one of the most used to model masonry structures is that which considers the blocks as rigid and assumes the contact forces on each block to be proportional to the inter-penetration between blocks, which is determined using the input contact stiffness.

The use of rigid blocks can be justified because the majority of damage was observed to occur due to inter-block displacements and because rigid blocks significantly reduce computation time.

The joint properties required for analysis are: the elastic shear stiffness, the elastic normal stiffness, cohesion and friction angle, tensile strength, dilatancy angle and density of blocks. Furthermore, numerical dynamic analyses require also definition of the damping ratio. It is important to note that, in the case of historical masonry, the poor quality of mortar joints justifies the assumption of cohesion and tensile strength null.

The value of joints stiffness *k* can be calculated by taking into account the modulus of elasticity of mortar (E_{mortar}) and stone (E_{stone}) as the following expression:

$$k = \left(\frac{s}{E_{stone}} + \frac{t}{E_{mortar}} \right)^{-1}$$

where *s* is the joint spacing and *t* is the thickness of the mortar layer. The friction angle can be given a very high value to ensure that no sliding would occur. By gradually lowering its value, it is possible to identify the threshold below which sliding occurs. The sensitivity of the response to different modelling parameters should be investigated case by case.

3.5 Modelling strategies for the different classes of architectonic assets

The present chapter deal with the possible application of the abovementioned models to the different classes of architectonic assets. Table 4.20 illustrates the cross-correlation between architectonic classes and damage modes, useful to address the choice of the most suitable model to be adopted.

DRAFT

Table 4.20.Correlation between type of buildings and damage classification.

DRAFT

Architectonic asset class		Damage class A	B	C	D	E	F	G	H	I
A	A1 - palaces	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
	A2 - castles	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
	A3 - religious houses	🟥	🟧	🟧	🟧	🟨		🟧	🟧	🟧
	A4 - caravansaries	🟥	🟧	🟧	🟧	🟨		🟧	🟧	🟨
	A5 - collective buildings	🟥	🟧	🟨	🟧	🟨		🟧	🟧	🟨
B	B1 - churches	🟧	🟥	🟧	🟧	🟨		🟧	🟧	🟧
	B2 - mosques	🟧	🟥	🟧	🟧	🟨		🟨	🟧	🟧
	B3 - baptisteries	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B4 - mausoleums	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B5 - hammam	🟧	🟥	🟨	🟧	🟨			🟧	🟧
	B6 - modern theatres	🟧	🟥	🟨	🟧	🟨		🟧	🟧	🟨
	B7 - markets and bazaars	🟧	🟥	🟨	🟧	🟨		🟨	🟧	🟧
	B8 - industrial buildings	🟧	🟥		🟨	🟨		🟧		
C	C1 - towers	🟧		🟥		🟨		🟨		
	C2 - bell towers	🟧	🟨	🟥	🟧	🟨		🟨		
	C3 - minarets	🟧	🟨	🟥	🟨	🟨		🟨		
	C4 - lighthouses	🟧	🟨	🟥		🟨		🟨		
	C5 - chimneys	🟧		🟥						
D	D1 - triumphal arches				🟥	🟧			🟨	
	D2 - aqueducts		🟨	🟨	🟥	🟧			🟧	
	D3 - bridges		🟨	🟨	🟥	🟧			🟧	
	D4 - cloisters				🟥	🟨			🟧	
E	E1 - fortresses					🟥		🟨		
	E2 - defensive city walls					🟥				
	E3 - Roman and Greek theatres			🟨	🟧	🟥	🟧			
F	F1 - columns						🟥			
	F2 - trilithes						🟥			
	F3 - obelisks						🟥			
	F4 - archaeological ruins				🟧	🟨	🟥			
	F5 - Greek temples						🟥			
G	Historical centers	🟧	🟧	🟨	🟨	🟨		🟧	🟧	

🟨 Occasional behaviour
🟧 Possible behaviour
🟥 Prevailing behaviour

Damage classes: A- Damage to in-plane loaded walls; B- Damage to out-of-plane loaded walls; C- Damage to monodimensional masonry elements; D - Damage to in-plane loaded arches (or vaults); E- Local damage of masonry; F- Rocking of single or multiple blocks; G - Damage to roofs and floors; H - Drift of vaults in their horizontal plane; I - Damage to domes

DRAFT

3.5.1 Class A (Palaces)

Buildings belonging to class A are typically characterized by a so-called "box behaviour". This means that the structure tends to behave as a whole and the collapse mainly derives from the in-plane damage of a number of macroelements (masonry walls and horizontal diaphragms) and masonry structural elements (piers and spandrels).

This type of behaviour may be attributed to those buildings made of vertical walls and horizontal diaphragms, with relatively good connections and relatively low amount of arched and vaulted systems. Vertical walls are usually rather slender (two dimensions prevailing on the third, the thickness) while diaphragms are stiff enough in the horizontal plane to convey seismic forces to masonry walls.

In this type of buildings, inertial forces are transferred through diaphragms to masonry walls proportionally to their stiffness and strength. In general, due to geometry and limited tensile strength of the material, stiffness and strength of masonry walls subjected to in-plane or out-of-plane forces are very different (the in-plane loaded walls being more stiff), the walls parallel to the prevailing direction of the seismic force are subjected to higher seismic forces, acting in their plane. Thus, the collapse of the building derives from the collapse of these latter macroelements. Out-of-plane loaded walls, which are less resistant but carry lower seismic forces, may collapse for loss of equilibrium (refer to damage class B) only in case of particular defect of the construction (excessive slenderness of a wall, lack of connection between perpendicular walls, presence of thrusting vaults).

For the structure belonging to Class A, the seismic assessment requires referring to a global 3D Continuum Constitutive Laws Models (CCLM) or Structural Element Models (SEM); in this case, the seismic behaviour is *described by a single capacity curve*. In general terms, it can be stated that (CCLM) allows us to simulate the response of masonry structure with a degree of accuracy greater than (SEM); however, the CCLM employment in practice presents some difficulties. In fact, CCLM requires a large computational effort that may become unsustainable for complex and large structures; moreover, in general its application requires the definition of a large number of mechanical properties that not always are available (so increasing the uncertainties in the seismic assessment). On the contrary, SEM allows to perform nonlinear analyses (both static and dynamic) with a reasonable computational effort and to have a "better" control of the response due to the limited number of mechanical properties involved.

For this reason, SEM (Structural Element Models) are the main models to be used for the global analysis of masonry structures made up of walls with regular openings, describing with adequate accuracy the in-plane behaviour of single structural elements.

SEM is particularly suitable for these buildings, because it allows to model complex assets with limited computational efforts, adopting nonlinear approaches both static and dynamic. Despite this, it is evident that in the case of very simple geometrical configurations, the seismic assessment is also possible performing non-linear analyses through a 3D CCLM.

In the field of SEM, the "equivalent frame" approach is the most widely diffused. It considers the walls as an idealized frame in which deformable elements connect rigid nodes. This latter represents one of the crucial points for such type of models; indeed, in case of very irregular opening pattern or complex walls configuration this idealization may result quite conventional. In such cases, in order to improve the reliability

DRAFT

of the seismic assessment and to take into account the uncertainty related to the frame idealization, two paths should be followed:

- carrying out a sensitivity analysis by considering it as an epistemic uncertainty (thus, treated by the logic tree approach);
- using different kind of models in order to support and define adequately the modelling criteria of the "equivalent frame" (for example, nonlinear analyses on CCLM of a wall could show the structural elements stiffer or weaker, consequently could address the definition of spandrels, piers or rigid nodes).

More in general, the adoption of other type of models – like as the CCLM and MBM models – for supporting and deepen some specific aspects of the structural response, could be advisable in some cases.

Moreover, it is worth noting that the adoption of a 3D model implicitly refers to the seismic assessment of the building as a whole; in fact, the analyses performed allow to involve the total mass of the structure in the seismic loads. Therefore, an exhaustive seismic assessment would require also the verification on the possible occurrence of local out-of-plane mechanisms.

3.5.1.1 Issues on the modelling of the local mechanisms for class A

It seems useful remarking that, based on damage survey after earthquakes, seismic performance assessment commonly refers to two different type of responses:

- the global seismic response of the structure, that in the case of Class A is associated to the in-plane response of walls and to the activation of a box-behaviour;
- the local response of single parts, usually subjected to out-of-plane mechanisms.

It is evident that the exhaustive seismic verification should take into account both of them, which can be analysed separately.

Local mechanisms are generally associated to different out-of-plane failure modes that may occur in a building of the class A. Generally, they may be analysed through macro-blocks models (MBM), using the limit analysis.

As regards the identification of structural masonry portions to be modelled, it is interesting to highlight that for MBM the shape of each block is defined "a priori" on the basis of: the recurrent crack patterns observed in post-earthquake surveys, the constructive details and aseismic devices of the structures (for example the presence of tie rods, the connections between the walls and walls and floors).

In the case of the assets of class A, for the definition of the macroelements it is possible to follow the survey form adopted by the Italian Department of Civil Protection "Scheda per il Rilievo del Danno ai Beni Culturali - Palazzi". Figures 104-110 show some of the macroelements and relative collapse mechanisms listed in the form.

DRAFT

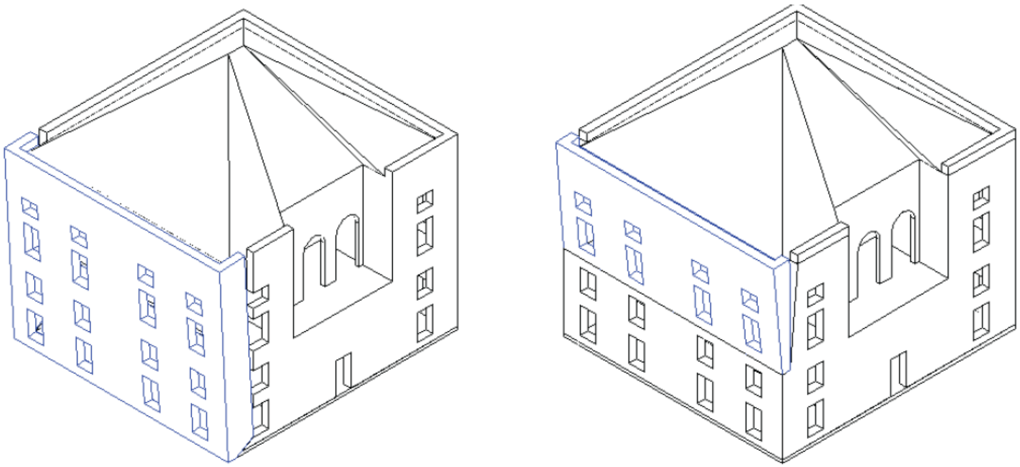


Figure 4.104. Overturning of the walls

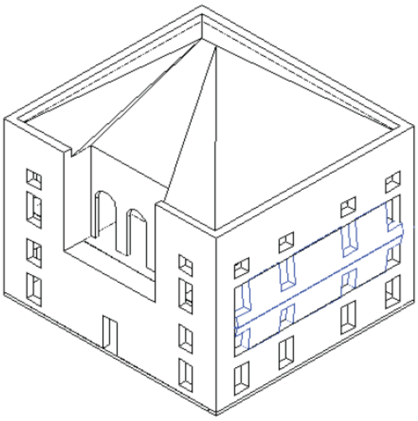


Figure 4.105. Vertical instability of the walls

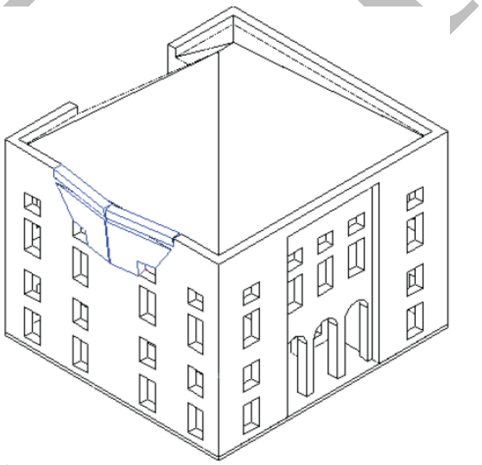


Figure 4.106. Bending collapse of the walls

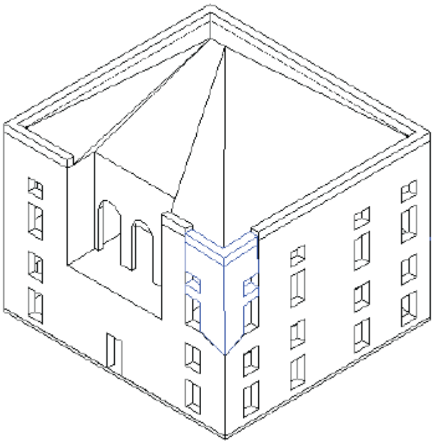


Figure 4.107. Overturning of the corner

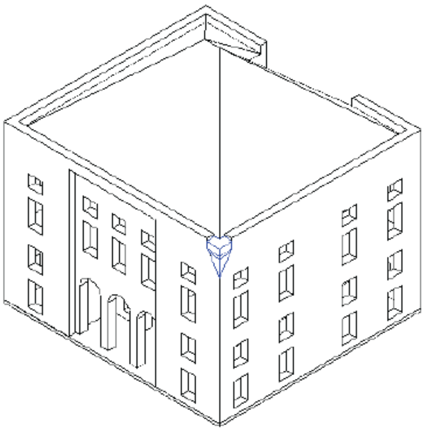


Figure 4.108. Overturning of the attic spandrel and gable

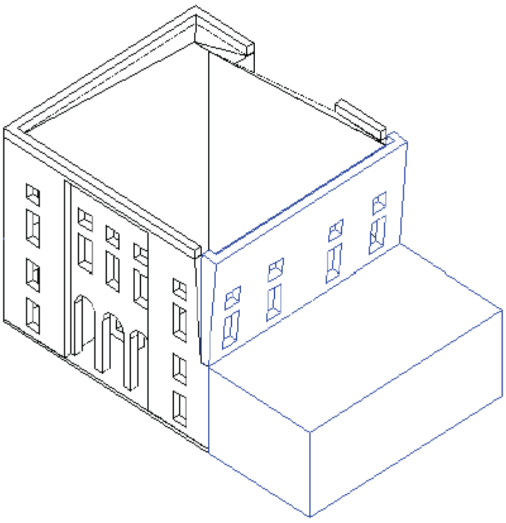


Figure 4.109. Damage due to the elevation irregularity

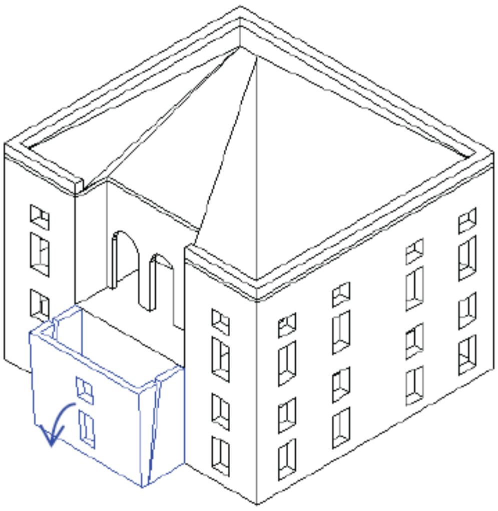


Figure 4.110. Damage to the annexed buildings

DRAFT

3.5.1.2 Issues on buildings in aggregate or without a clear box-behaviour

Buildings in the aggregate

In many cases the palaces are not isolated but they are part of a more complex aggregate. The SEM of all aggregate is onerous, so the interactions with other buildings are considered by modelling specific boundary conditions (that simulate the stiffness of the other buildings) in the SEM model of the analysed structure. However, this approach is complex and conventional.

For this reason, in order to assess the seismic behaviour of the building in the aggregate (then modelling the whole aggregate), it is advisable the use of simplified mechanical model (according to the procedure proposed in the Italian Guidelines for evaluation and mitigation of seismic risk to cultural heritage. Directive of the Prime Minister, 12/10/2007). The results of this simplified analysis should be compared with those obtained from the detailed analysis of the single structure, in order to be aware if the seismic performance improves or worsens (considering the contribution of the aggregate).

It is important to note that the interactions between different parts of the same aggregate can create different effects (that is also reflected in differing modelling criteria) in relation to:

- the position of the asset in the aggregate (inner or at the corner). In this case, if the seismic analysis is performed not considering the adjacent structures, the results will be in favour of safety if the building is located within the aggregate, vice versa if it is at the end;
- the difference in stiffness (due to different number of storey or the area in plan) between the assets object of the analysis and the rest of the aggregate. In fact, if the building is stiffer than the rest of the aggregate, the analysis can be performed without considering the contribution of the aggregate;

- the connection of the asset with adjacent ones. If the connection is realized with a flexible structural element (for example an archivolt) the modelling of the whole aggregate is not essential, but particular attention should be paid to stress concentration on the connection element.

Buildings without a box-behaviour

The seismic assessment of the buildings belonging to class A with very flexible floors and poor connections among walls can be performed analysing the in-plane response of each single wall. Thus the problem of modelling moves from the global structure to the scale of the single walls and the seismic assessment can be employed by both SEM and CCLM. In this case the CCLM is practicable, as the model of a single wall is simpler than the model of the global structure.

3.5.2 *Class B (churches and religious buildings)*

The buildings, belonging to class B, are particularly prone to the out-of-plane damage. This is typical of those buildings in which walls are slender (due to the absence of horizontal diaphragms) and in which connections between structural elements are weak (lack of tie-rods, weak interlocking between walls …).

In fact, usually in the complex structures of the class B each wall or a portion of the building (macroelement) are characterized by autonomous and unitary structural behaviour under seismic action (case a) (Figure 4.111).

However, in some cases the seismic response of the complex assets of class B cannot be attribute to the macroelements approach, because of the geometry and of the prevalence of a global response under seismic action (case b).

So, in the first cases (case a) buildings made by a set of macroelements which exhibit an almost independent behaviour, the seismic response is generally studied directly at the scale of the single macroelement (Doglioni, 1994; Lagomarsino, 2004). Thus, the proper model for the nonlinear verification and the definition of the damage level is chosen for the single macroelement. In this sense the seismic assessment of the whole structure can be performed through a combination of the pushover curves obtained by the independent analysis of each macroelement.

On the contrary in the latter cases (case b), buildings with a global behaviour, the seismic assessment requires referring to a 3D model. As the proper model must be selected to describe the whole structure, the choice of modelling tends to focus on the 3D CCLM, particularly suitable for modelling complicated geometry (for example characterized by curved surface, in general conditions of loading and constraints) and, rarely, on the 3D SEM.

DRAFT

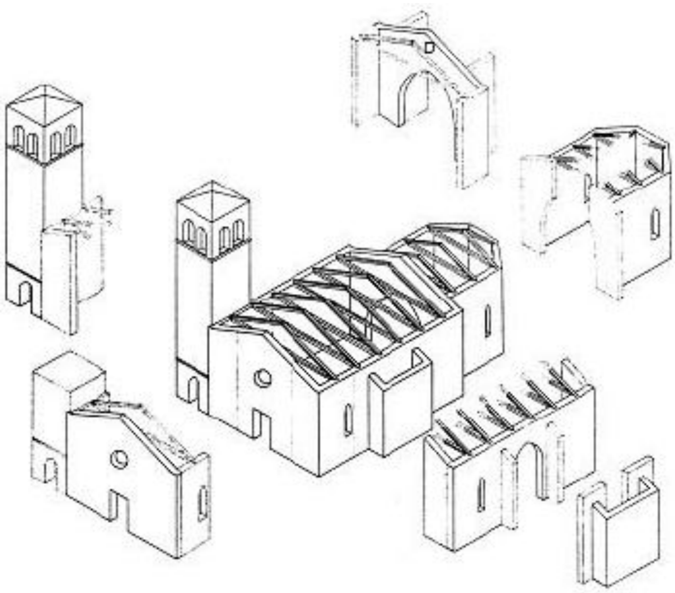


Figure 4.111. Example of the macroelements of a church (subclass B1)

Despite the aforementioned issues, it is important to note that also for the buildings made by a set of macroelements the combined use of a 3D CCLM should be very appropriate for the seismic assessment; in fact, it could be useful in order to:

- know if the structure shows in such a clear way the behaviour for parts of the buildings. In the most cases the seismic behaviour of the buildings belonging to class B is shown after the observation of the structure or past damages, but sometime the seismic response of complex buildings is not so obvious;
- identify accurately the macroelements (if it is possible to subdivide the structure through the macroelements), as a function of the asset plan configuration, the structural and constructive details. The deep knowledge of the asset and the damage observed are essential tools to address this issue;
- direct the choice of different mechanisms to be analysed for each macroelement;
- determine the loading redistribution among the macroelements.

3.5.2.1 Consideration on the modelling of the macroelements

In cases of buildings certainly made by a set of macroelements, the seismic response is generally studied directly at the scale of the single macroelement. So, each portion can be analysed through the different above-mentioned models.

The choice of the most suitable model is a function of:

- the geometry of the macroelement, in particular the distribution of the openings in the masonry walls;

DRAFT

- the prevailing seismic response (the in-plane or out-of-plane response), that in turn depends on the macroelement form, the external constraints and the analysis direction.

It is highlighted that the same macroelement, depending on the seismic direction, can be analysed with two different models.

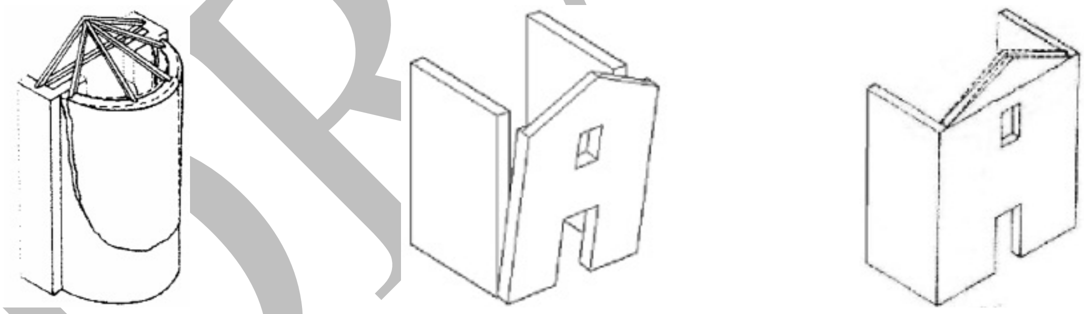
In reference to the models previously described, it is pointed out that: the MBM (Macro-Block Models) is the main tool for the seismic assessment of these portions, but is not the only possible choice, especially for the in-plane response. In fact, in this latter case, the CCLM (Continuum Constitutive Laws Models) and the SEM (Structural Elements Models) could be employed. The CCLM is suitable in case of both irregular geometry configuration and uneven disposal of the openings in the front, since the idealization of the equivalent frame would be not appropriate, vice versa, in case of regular geometry of the macroelement, also SEM can be used.

In some cases, quite rarely, the DIM (Discrete Interface Models) may be used.

It is evident that, before applying the different models, the identification of the macroelements is necessary. They can be determined by surveying the geometry, the connections to the rest of the asset and the damage occurred during the past earthquakes. Once defined the macroelements (with their configurations), the damage modes and the collapse mechanisms must be recognised. In many cases a single macroelement is chosen because its seismic response is considered as the most representative of the behaviour of the whole building.

For the determination of church macroelements (subclass B1), is possible to follow the survey form adopted by the Italian Department of Civil Protection (D.P.C.M. 23/02/2006). Figure 4.112 shows some of the macroelements and relative collapse mechanisms listed in the form.

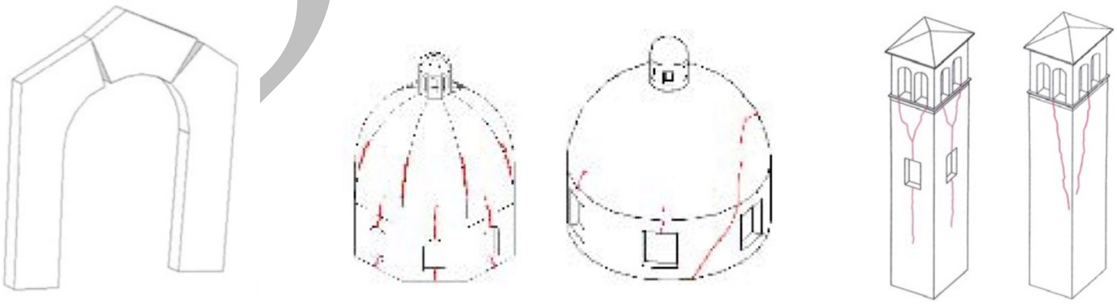
DRAFT



Apse

Façade

Gable of the façade



Triumphal arch

Dome and Tiburio

Bell tower

Figure 4.112. Example of dome macroelements of a church (subclass B1)

The macroelement façade of Figure 4.113 can be analysed with a MBM. This is due to its geometry, in particular for the presence of pillars, arches and vaults, and to the hypothesized collapse mechanism, with the formation of 4 hinges at the top and 4 at the base of the pillars.

Another type of macroelement façade (Figure 4.114), instead, can be assessed through a structural elements model (SEM). The in-plane seismic response of this macroelement suggests the use of SEM, as the presence of regular piers and openings facilitates the idealization of the equivalent frame for this model.

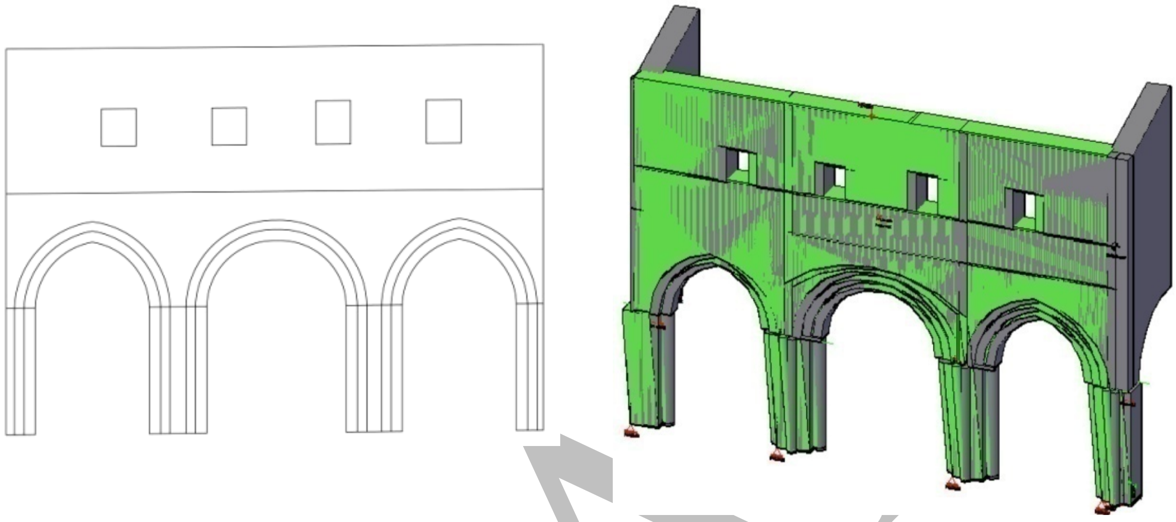


Figure 4.113. The MBM of the macroelement façade

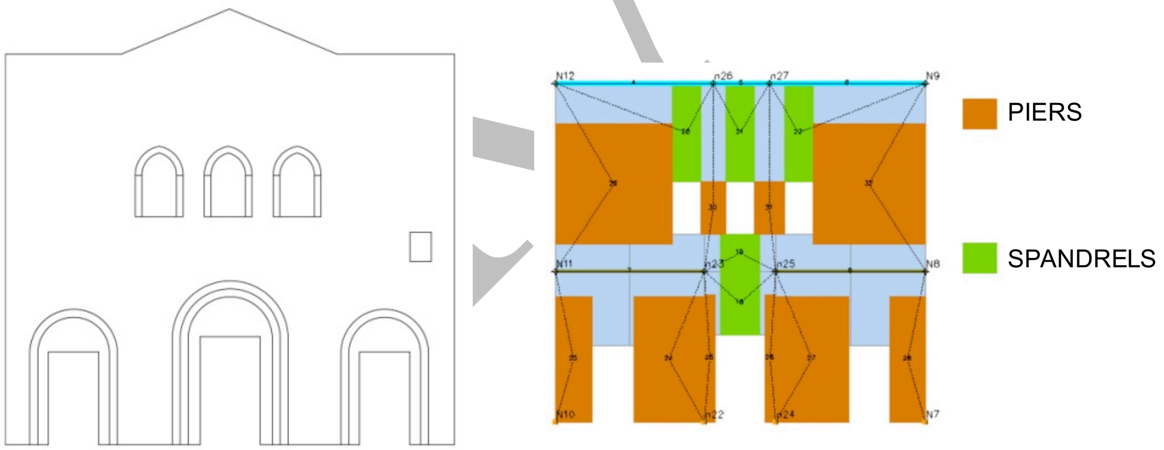


Figure 4.114. The SEM of the macroelement “internal façade”

3.5.2.2 Consideration on the 3D CCLM linear model

In case of buildings belonging to class B with a global behaviour, the seismic assessment requires referring to a 3D model. The most suitable model for the seismic assessment of these types of structures is the CCLM.

However, in case of huge buildings it is very difficult to adopt nonlinear constitutive laws for modelling masonry material; so it is necessary to refer to linear models.

The model is aimed to:

- understand the structural behaviour of the asset and ensure it is rationale to adopt a macroelements approach, when this is not so obvious from a qualitative point of view. In the cases of buildings that show a global behaviour, the 3D CCLM suggests criteria and model to be adopted for the assessment;
- support the definition of the macroelement and address the choice of plausible mechanisms to be examined. This issue is realised through the analysis of the stress state, because hinges will form where peaks of tensile stresses occur;
- help to define the loading distribution among the macroelements that compose the asset. The loading distribution is an important aspect, which in general is adopted in a conventional way, as a function of the masses that weigh directly on each macroelement. However, this choice can be arbitrary and, sometimes not on the safe side, especially when the structures have undergone a strengthening intervention, which made the connections between the structural elements more efficient and the horizontal diaphragms rigid (in order to improve the box behaviour of the structure).

Issues on the definition of 3D CCLM models

The information acquired through the knowledge phase (in particular the geometry of the asset, the connections between the structural elements and the mechanical properties of masonry) give the features of the CCLM global model.

It is important to note that the results of the CCLM depend on the choices made for the modelling (element types, stiffness properties and connection between the structural elements); for this reason, a detailed knowledge phase is essential and the sensitivity analysis may be useful for directing the modelling choices.

In particular, in case of linear models, the results in terms of stress concentrations are very sensitive to modelling choices and their interpretation must take into account this drawback.

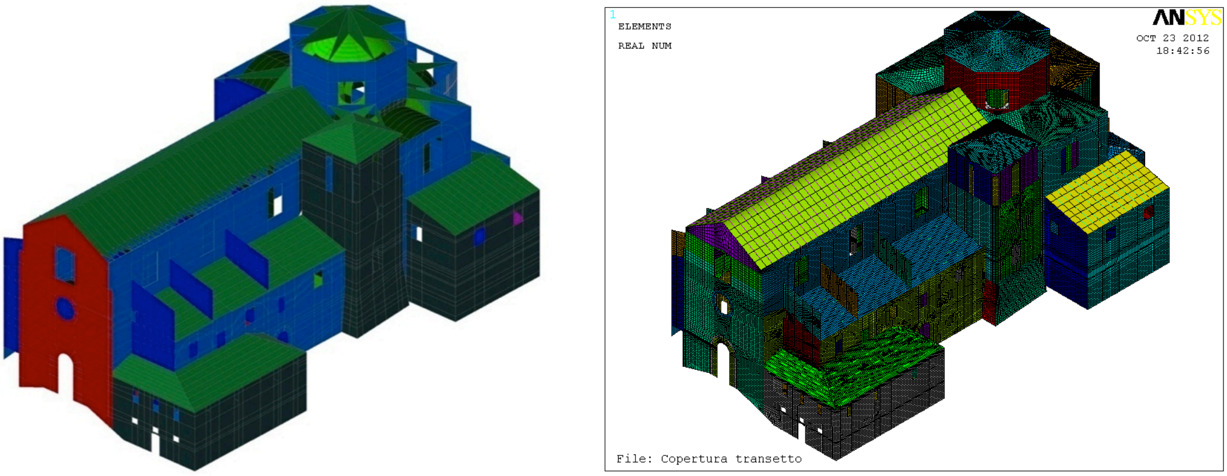
As a function of the characteristics of each structure it will be necessary to adopt the appropriate element types. For example, in case of massive structures it is possible to use 3D solid elements, on the contrary the masonry walls, vaults and domes should be modelled through 2D shell elements (with bending and membrane capabilities), for which both in-plane and normal loads are considered.

With regard to the stiffness properties of the structural vertical elements, the parameters shall be calibrated as a function of the masonry types. In the case of horizontal structures, the detailed modelling of vaults and domes allows to take into account the stiffening contribution due to their shape and geometrical proportion (e.g. rise-to-span ratio); the roof and the other types of floors, as an alternative to the detailed modelling of each structural element, could be modelled like slab provided with an equivalent stiffness.

The 3D CCLM model of the buildings belonging to class B are generally very complex, so it is convenient to start from a geometrical model, where the mid-plane of structural elements has been defined. By drawings (plans, fronts and sections) it is possible to determine the thickness and height of each wall, the location of the openings and the roof, the definition of the curvature of arches and vaults.

DRAFT

Once the mid-plane has been defined, the 3D geometrical model of the structure must be realised (Figure 4.115a). Then, the geometrical model can be exported from the CAD environment and imported in the FEM software, with which it can be carried out both the CCLM modelling (Figure 4.115b).



a. b.

Figure 4.115. The three-dimensional geometrical model of Santa Mara Paganica Church (a); the CCLM (B)

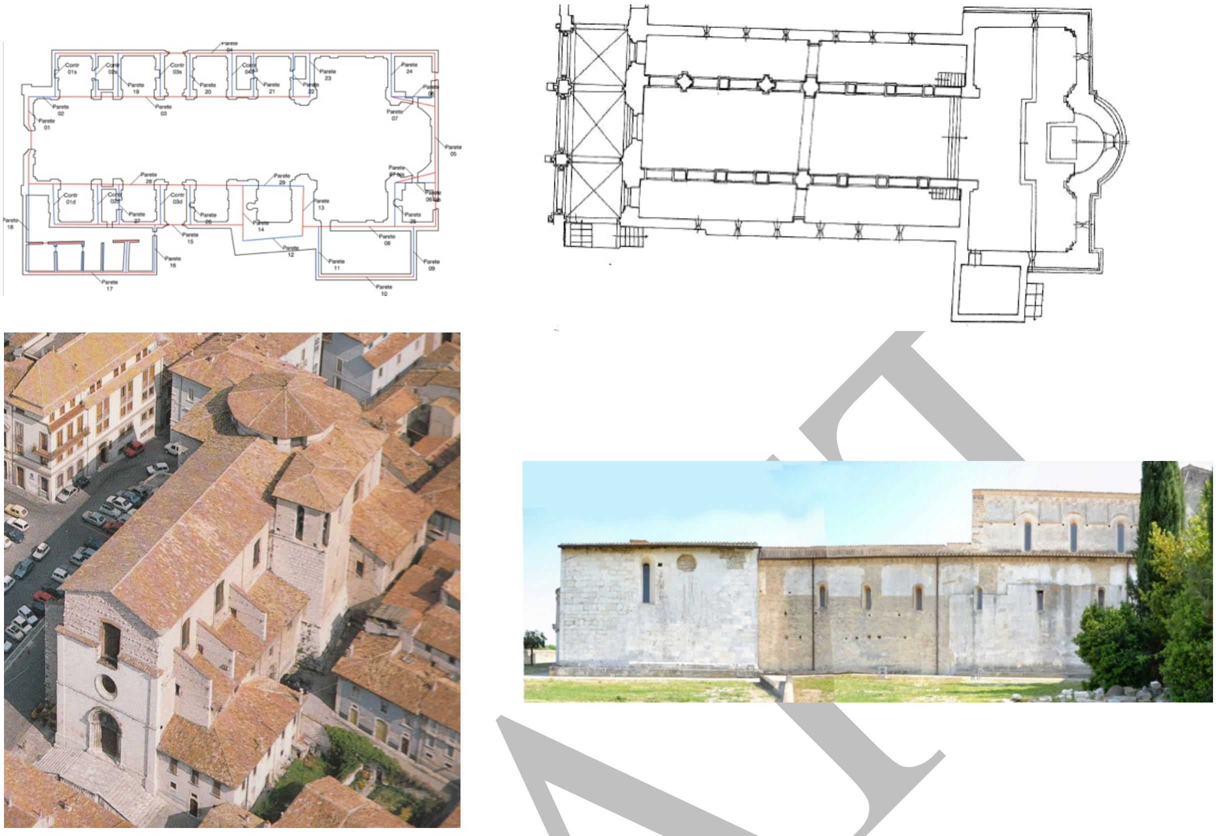
Use of the 3D CCLM model to understand the structural behaviour and supporting the choice of the most appropriate approach to be adopted (global or by macroelements)

Through the identification of the vibration characteristics of the CCLM model (natural frequencies and mode shapes), it is possible to understand if the asset has a macroelement seismic response.

If the 3D CCLM linear model shows mainly a global behaviour in the seismic response, with a strong interaction among the different portions of the structure (as the case study of Figure 4.116a), the seismic assessment through the subdivision into macroelements seems to be not justified. In this case, the CCLM of the global structure is the more suitable model for the assessment of the structure. Exceptionally the SEM may be employed. For this building it is essential to perform some local seismic assessment, when the damage survey or the CCLM highlights some vulnerable macroelements or structural portions that show an independent behaviour compared with the other parts of the structure.

If the macroelements division is evident from the configuration of the structure (as for the church in Figure 4.116b) or 3D CCLM linear model shows that each part of the fabric is characterized by a prevalent autonomous structural response, the seismic assessment can be carried out following the macroelement approach.

DRAFT



a. b.

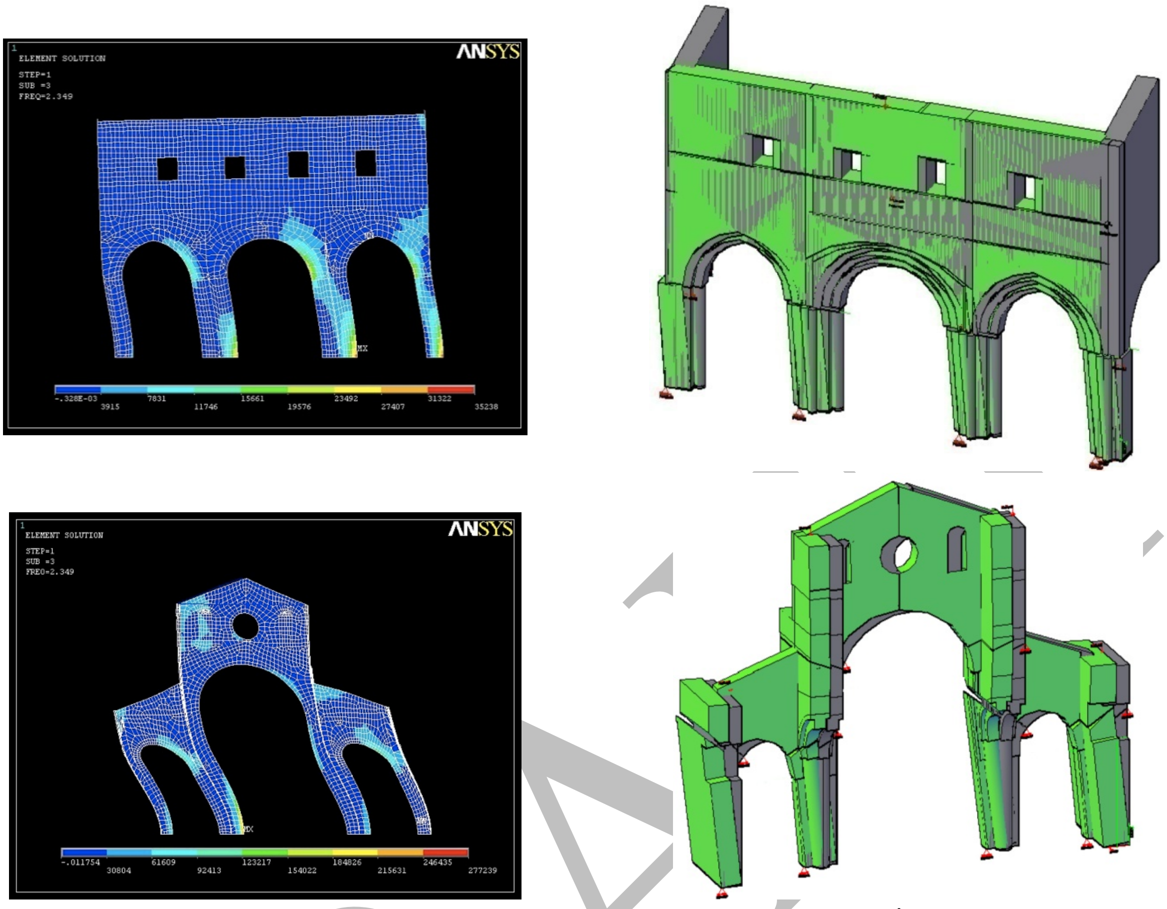
Figure 4.116. Two different churches plans

Use of CCLM model to support the definition of macroelements and kinematisms to be analysed

If the building can be studied through a macroelement approach, the second step concerns primarily the identification of the macroelements in the CCLM model and consequently the choice of hypothetical mechanisms to be examined. This issue is realized through the analysis of the stress state, identifying the stress peaks where the first hinges will form, thus addressing the choice of collapse mechanism.

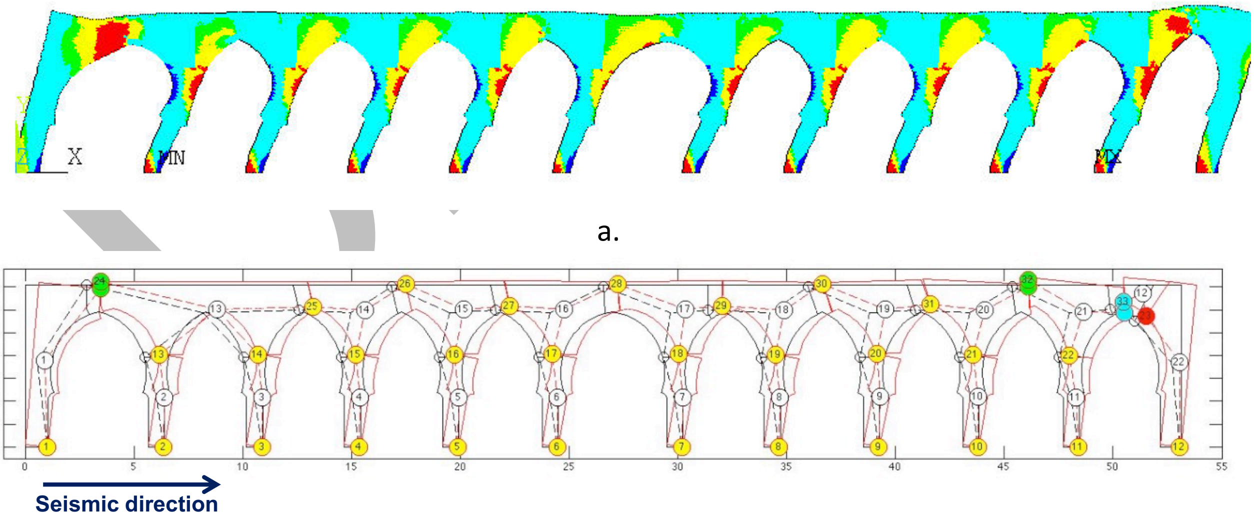
If linear analyses are performed, only the information concerning the first hinges are obtained (Figure 4.117). Instead, in some cases it might be useful and not particularly onerous to carry out nonlinear analyses on CCLM model, for verify the evolution of the mechanisms in the nonlinear branch (Figure 4.118).

DRAFT


ELEMENT SOLUTION
STEP=1
SUB =3
FREQ=2.349
ANSYS
-.328E-03
3915
7831
11746
15661
19576
23492
27407
31322
35238
ELEMENT SOLUTION
STEP=1
SUB =3
FREQ=2.349
ANSYS
-.011754
30804
61609
92413
123217
154022
184826
215631
246435
277239

a. b.

Figure 4.117. The stress states in the CCLM: (a) and the collapse mechanisms in the MBM (b) for the macroelements façade and triumphal arch.


X
MN
MX
0
5
10
15
20
25
30
35
40
45
50
55
Seismic direction

a.

b.

Figure 4.118. The stress state in the CCLM (a) and the collapse mechanism in the MBM (b) for the longitudinal macroelement of the Great Mosque of Algiers (subclass B2).

3.5.3 Class C (towers)

Buildings belonging to class C are slender structures that suffer large bending effects owing to the high level of the centre of gravity of their masses. Due to their height, such structures are typically subjected to high vertical compressive stresses even under gravity loads; bending derived from seismic actions produces further increases leading the structure to an overall failure in compression. Moreover, in some cases, a prevailing shear response may occur, which can be interpreted by referring to the beam theory.

These structures may be of various shape (square, circular or polygonal) but usually are squared structures regular in their height, with limited and small openings. Floors are presents at different levels or, in some cases, are totally absent (Chimneys C5); in this latter case the structure is usually very slender.

In the case of bell towers, they are characterized by a set of openings concentrated on their top. From the seismic point of view, the presence of these openings represents a vulnerability factor, since often damage in concentrated in this part of the structure. The need to reach the top of the structure determines the presence of staircases and, sometimes, intermediate floors, which usually have a strengthening effect.

The assets belonging to class C are generally simple structures (both from a geometrical and constructive point of view), therefore CCLM models are the more accurate and applicable in this case. However, CCLM remains complex due to the use of appropriate nonlinear constitutive laws.

DIM is not suitable, as it would require an excessive number of elements, with the exception of structures formed by the overlapping of large dimension blocks.

Even the MBM is not particularly useful for the global response of these structures, since the elastic deformability (fundamental in the analysis of the seismic response of this type of assets) would not be considered. But MBM may be used in the analysis of the local mechanisms related to single parts of the structure (such as the belfry in the bell towers).

Thus SEM is the most suitable tool for the global assessment (Figure 4.119). In the case, of this type of structures it can be used as using nonlinear elements of appropriate constitutive features.

For squat towers, the four walls of the structure can be modelled following the traditional approach of the “equivalent frame”. Vice versa the global behaviour of slender towers is not ascribable to the in-plane response of the four walls, so this type of structure should be modelled as an “equivalent nonlinear cantilever beam”.

DRAFT

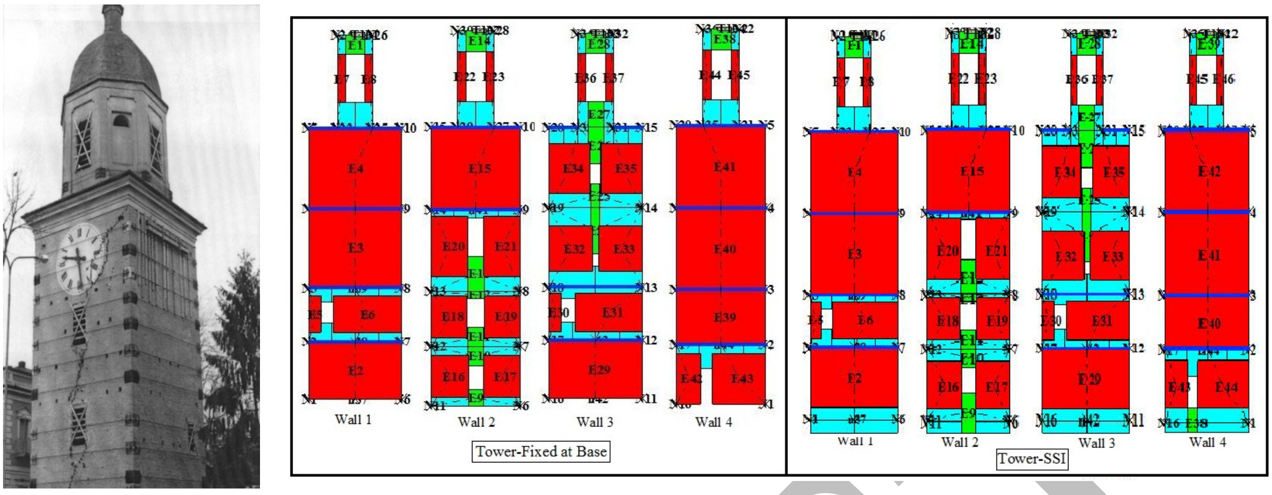


Figure 4.119. An example of bell tower modelled with SEM through the “equivalent frame” approach.

3.5.3.1 Consideration on the modelling of towers (C2)

The structures belonging to the Class C were usually built with high quality materials and good masons, but for their constructive details they have intrinsic vulnerability in respect of the earthquake, which favours the formation of local collapses of projecting or not connected portions.

The vulnerability is also influenced by the presence of pre-existing damage, for example due to vibrations caused by bells or the foundation problems. These vulnerabilities generate, statistically, a damage level greater than that of existing ordinary buildings.

In particular, the seismic behaviour of these structures is dependent on specific factors:

I. The slenderness;

II. The quality of the connections between the walls;

III. The presence of slender architectural element on the top (standing out elements or belfry);

IV. The possible presence of lowest adjacent structures, able to provide a horizontal constraint.

The slenderness (I) is a parameter highly variable: in fact, there are both very squat towers and bell towers of great slenderness. If the former can be considered as small constructions similar to those of Class A, the latter can be considered as monodimensional structures with cantilever behaviour.

The connections between the walls (II) are functional to ensure that slender structures behave like a cantilever fixed at the base, with a stiffness associated to the whole section wall (with a limit case of conservation of the transversal sections as flat) and not as a set of distinct walls.

As regard the issue (III), for example the belfry of the bell towers may be a particularly vulnerable element, because there are large openings that create slender pillars. Similar considerations are valid for the slender and standing out elements, often found on the top of the towers. Their vulnerability is primarily due to the low vertical load (associated only with its own weight), which provides a limited stabilizing effect in regard to the overturning.

DRAFT

Even more critical is the effect of the amplification of the seismic motion that occurs in the highest parts of the constructions.

Very frequently is the case of towers connected to the lowest adjacent structures (IV): bell towers joined with the church; towers that are part of a big fortress; towers included in the city walls.

The presence of these structures (walls, churches, palaces …) can be a constraint for the tower and its seismic vulnerability is influenced by the dynamic interaction between them. In fact, the lowest structures limit the slenderness of the tower, but at the same time originate points of stresses concentration.

The damage survey after recent earthquakes (Figure 4.120-4.121) showed many cases in which the interaction with the church generates cracks in the bell tower. Generally, shear cracks starting at the point of contact with the churches occur. After the Emilia earthquake, the most of bell towers constrained to the churches have showed this type of cracks.

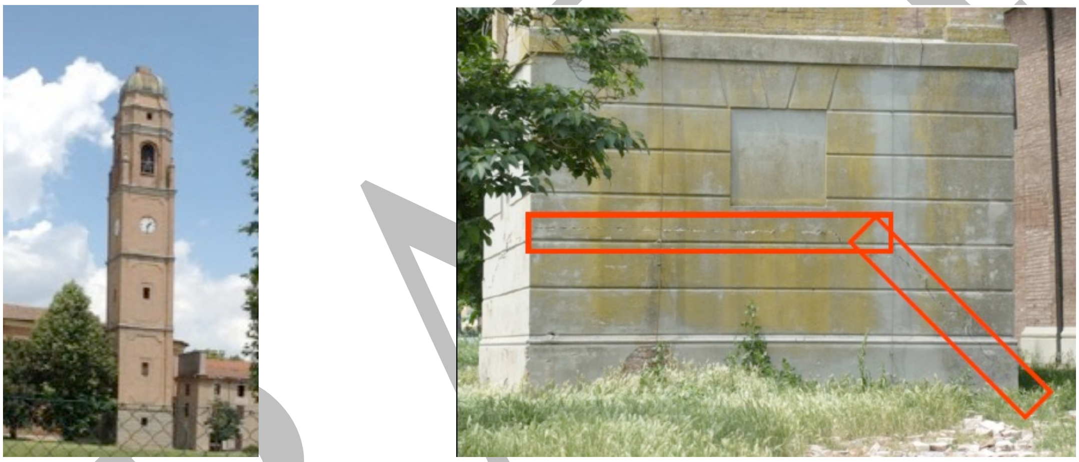


Figure 4.120. Isolated bell tower in the Emilia Romagna Region and the flexural cracks occurred during the earthquake of May 2012.

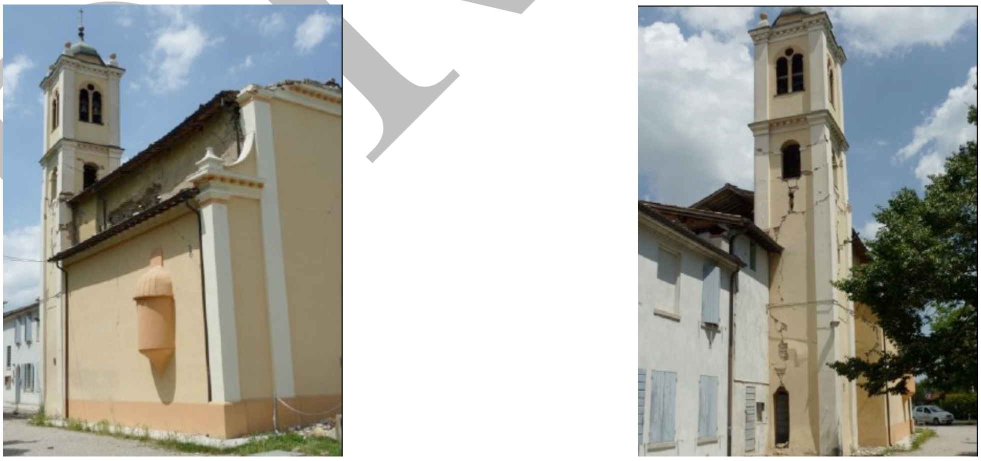


Figure 4.121. Bell tower connected to the church in the Emilia Romagna Region and the shear cracks occurred during the earthquake of May 2012

DRAFT

The different seismic behaviour of the isolated and constrained towers is shown by the observation of the post-earthquake damage but it is also confirmed by analyses on CCLM models of: isolated towers and towers connected to a wall (see Cattari et al. 2013 for further details). From these analyses, it is highlighted that the bending response is predominant in the case of the isolated tower, with maximum compressive and tensile stresses at the base (Figure 4.122).

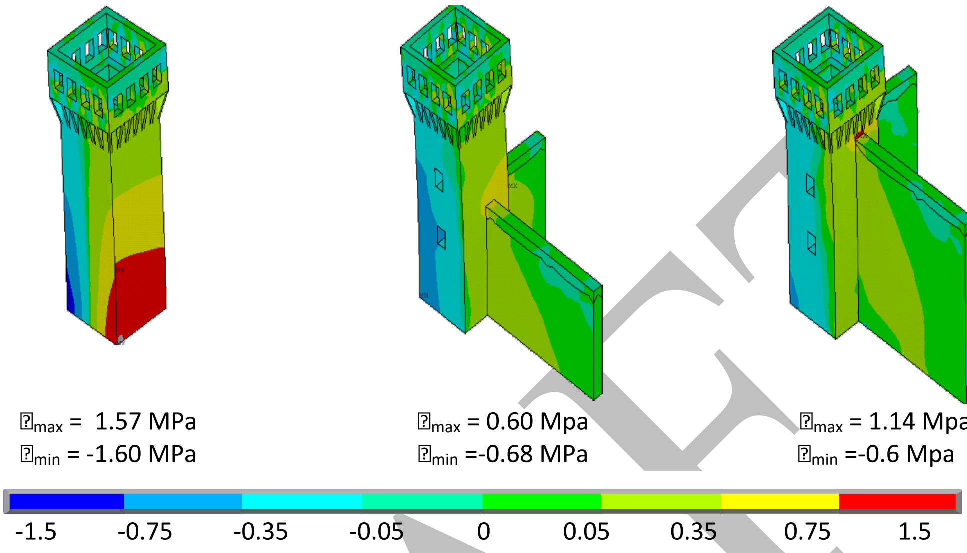

$\square_{max}$ = 1.57 MPa
$\square_{min}$ = -1.60 MPa
$\square_{max}$ = 0.60 Mpa
$\square_{min}$ =-0.68 MPa
$\square_{max}$ = 1.14 Mpa
$\square_{min}$ =-0.6 Mpa
-1.5
-0.75
-0.35
-0.05
0
0.05
0.35
0.75
1.5

Figure 4.122. Comparison between vertical stresses in the models, due to their weight and to the first mode contribution (Cattari et al. 2013)

It is also interesting to observe the shear stresses distribution in the walls of the tower (Figure 4.123). They are concentrated in the central portion because, considering the tower as a cantilever beam, the average shear stresses are maximum near the neutral axis. For the isolated tower, the maximum value is at the base, where the shear is greater, while when the fortress walls are well connected to the tower, the maximum value is observed at the top of these walls. They are significant in the whole first part of the body of the tower and tend to orientate toward the external corner near the base, where they meet the orthogonal wall.

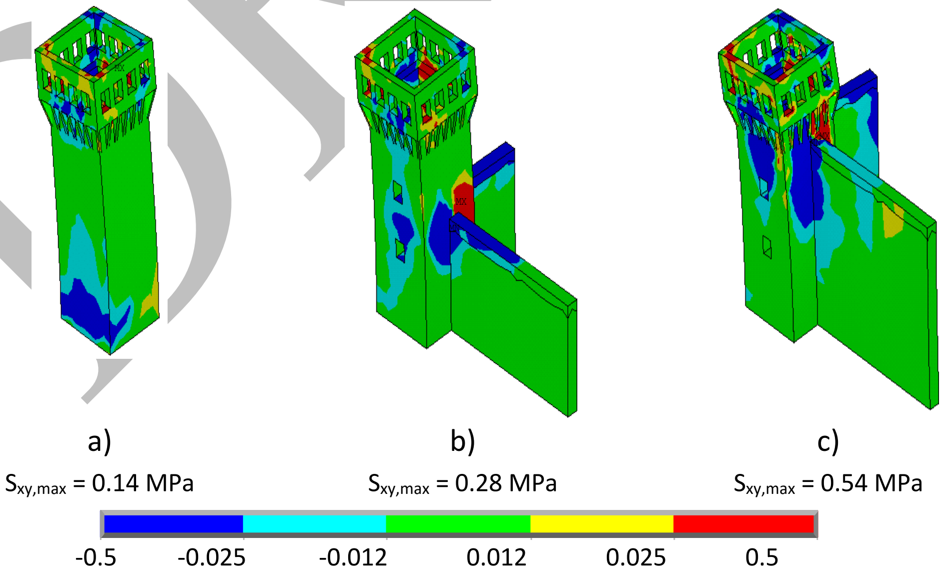

a)
b)
c)
$S_{xy,max}$ = 0.14 MPa
$S_{xy,max}$ = 0.28 MPa
$S_{xy,max}$ = 0.54 MPa
-0.5
-0.025
-0.012
0.012
0.025
0.5

Figure 4.123. Shear stresses in the models (Cattari et al. 2013)

DRAFT

3.5.3.2 Issues on the modelling of the local mechanisms the bell tower (C2)

The local assessment of the bell tower is carried out through the analysis of the collapse mechanisms of the belfry. The observation of the post-earthquake damage led to the identification of the most frequent collapses, highlighting how they occur for loss of equilibrium caused by the formation of cracks that transform the belfry into a kinematism of rigid blocks, which rotate or slide relatively one to the others.

These modes of damage suggest the use of MBM models, following the Limit Analysis according to a Kinematic approach. It is stressed that the MBM is possible if the quality of the masonry is fairly good. In fact, in the case of very poor mechanical parameters of the masonry the collapse occurs for disintegration of the masonry material. A behaviour not ascribable to a rigid blocks system was also observed in the case of belfry made of squared stone blocks without mortar.

In particular, it was noted that cracks are mainly located at the ends of the belfry pillars (Doglioni *et al.*, 1994); the main observed collapse mechanisms can be brought back to two types (Curti 2007, Figure 4.124 and 4.125):

1. Mechanism 1: the sliding outwards of one pillar and the rotation of the other one;
2. Mechanism 2: the rotation of the two pillars, with the formation of four hinges.

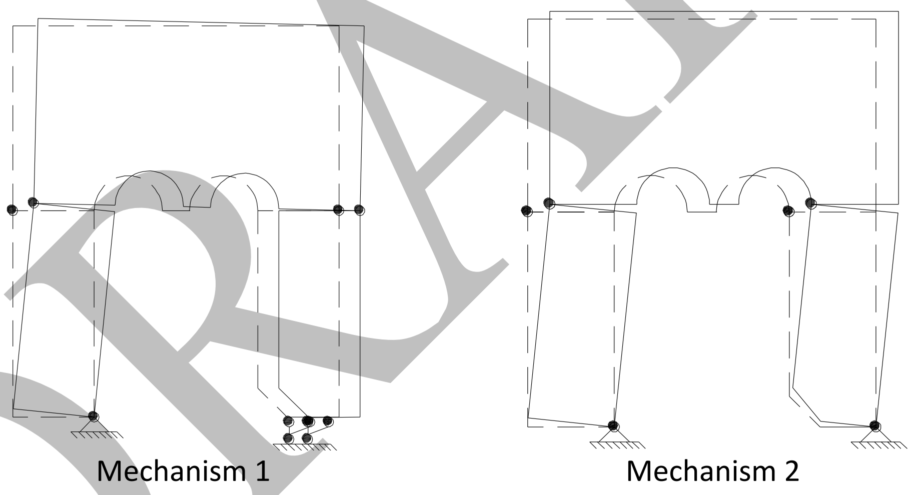


Figure 4.124. Two significant collapse mechanisms (Curti 2007)

DRAFT

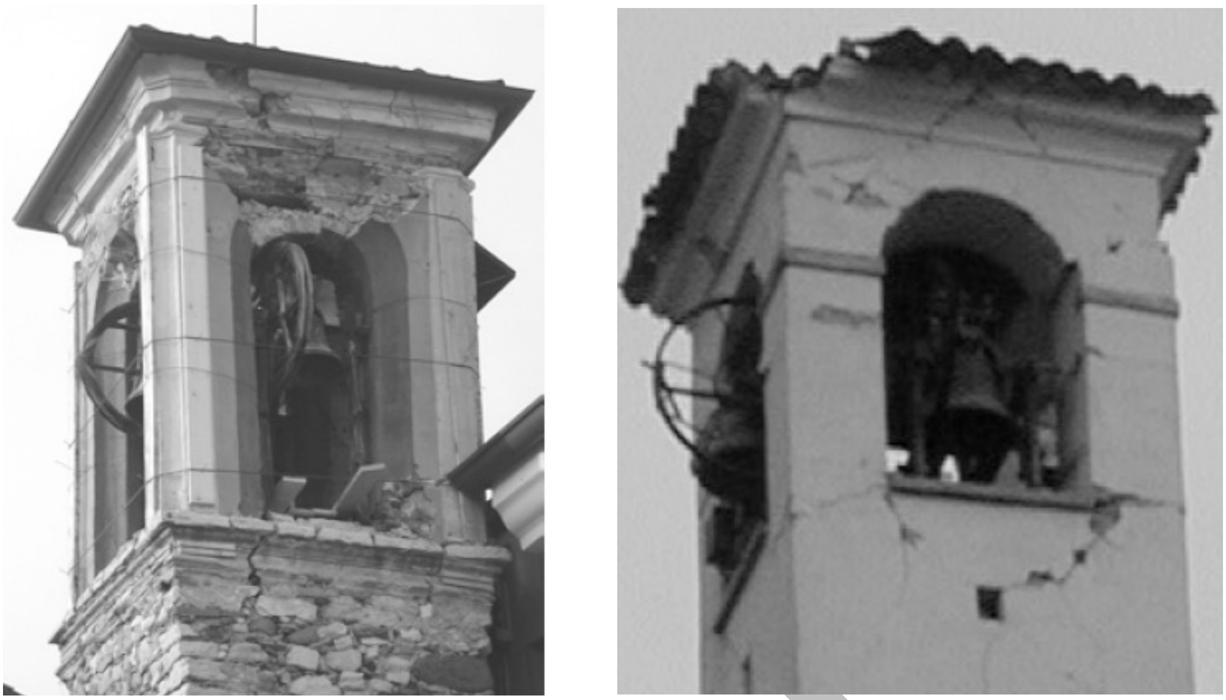


Figure 4.125. Some images of the recurring collapse mechanisms (Curti 2007)

In some cases, the interaction between belfry and tower induces a strong increase of compressive stresses at the base of the pillar and the damage of the belfry may also involve the tower, in particular the corners (Figure 4.126).

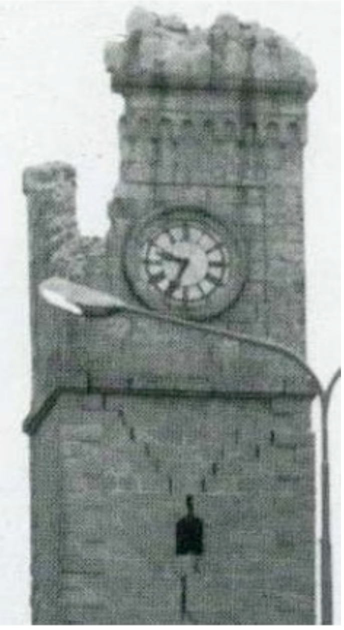


Figure 4.126. Damage at the corner of the tower (Doglioni et al., 1994)

Besides the out-of-plane mechanisms, in some cases the in-plane damage (characterized by shear cracks in the pillars) has been observed. This type of damage is possible if the structure has shown a box behaviour (good connections between the walls, presence of tie-rods …) and the pillars of the belfry are squat.

DRAFT

DRAFT

3.5.4 Class D (Triumphal arches, Vaults)

This class includes arches and vaulted structures, like triumphal arches, cloisters, masonry arch bridges. It is important to point out that only the structures that may be studied as independent are considered, while, when they are included in a bigger building (like a palace or a church), they have to be considered as macroelement and their response has to be analysed considering the relationship with the whole building.

In general, two main seismic responses can be considered:

- 2D mechanisms: they involve the development of a 2D response in the vertical plane of the structure, typical of arches or vaults with a single curvature (e.g. barrel vaults);
- 3D mechanisms: they are related to a response in the horizontal plane of vaults that involves a more complex damage mode (3D response of the asset) and is typical of double curvature systems (e.g. cross vaults) or system with an asymmetric constraints condition.

3.5.4.1 Considerations on modelling 2D mechanisms

The most representative assets that may undergo these kind of mechanisms are triumphal arches, bridges, aqueducts or also portions of aqueducts (archaeological remains that have lost the continuity with a more complex and extended structure).

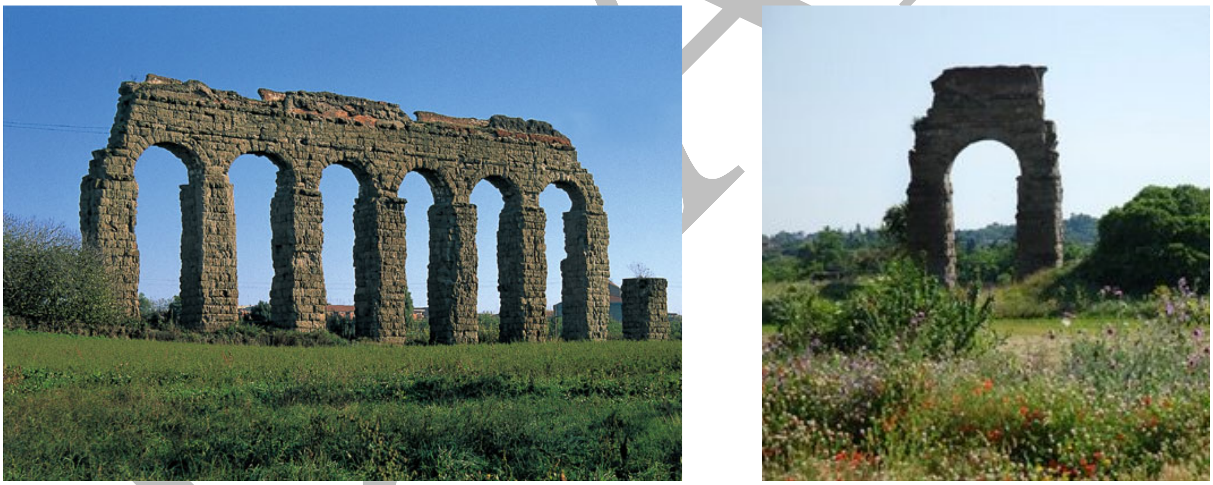


Figure 4.127. Archaeological remains of Roman aqueduct Claudia

These mechanisms usually involve the arch-piers system and occur by means of tensile cracks which turn the structure into a set of blocks. The structures can be readily simplified to two dimensions and they can be well studied using “Macro-Block Models (MBM)”.

As regarding the identification of structural masonry portions to be modelled, it’s important to highlight that for MBM the shape of each body is defined “a priori” on the basis of recurrent crack patterns observed in post-earthquake surveys.

In order to predict the behaviour of the system, a more refined way is the adoption of the CCLM approach. In particular, this modeling strategy can be useful for different aims:

- on the one hand, to gather the elastic behaviour of the structure (by performing an elastic linear analysis);
- on the other hand, to provide information about the state of damage and stresses distribution useful to define the most significant mechanism to be investigated through the MBM; in particular, the performance of an elastic linear analysis allows to individuate the position of the first plastic hinge, while by performing an inelastic non-linear analysis it is possible to follow the evolution of the kinematism.

However, it is important to note that the two modeling approaches can be used in a combined way, since the comparison between their respectively results are useful to better define the actual response of the structure.

Figure 4.128 shows an example of an arcade system modelled by using the two different strategies.

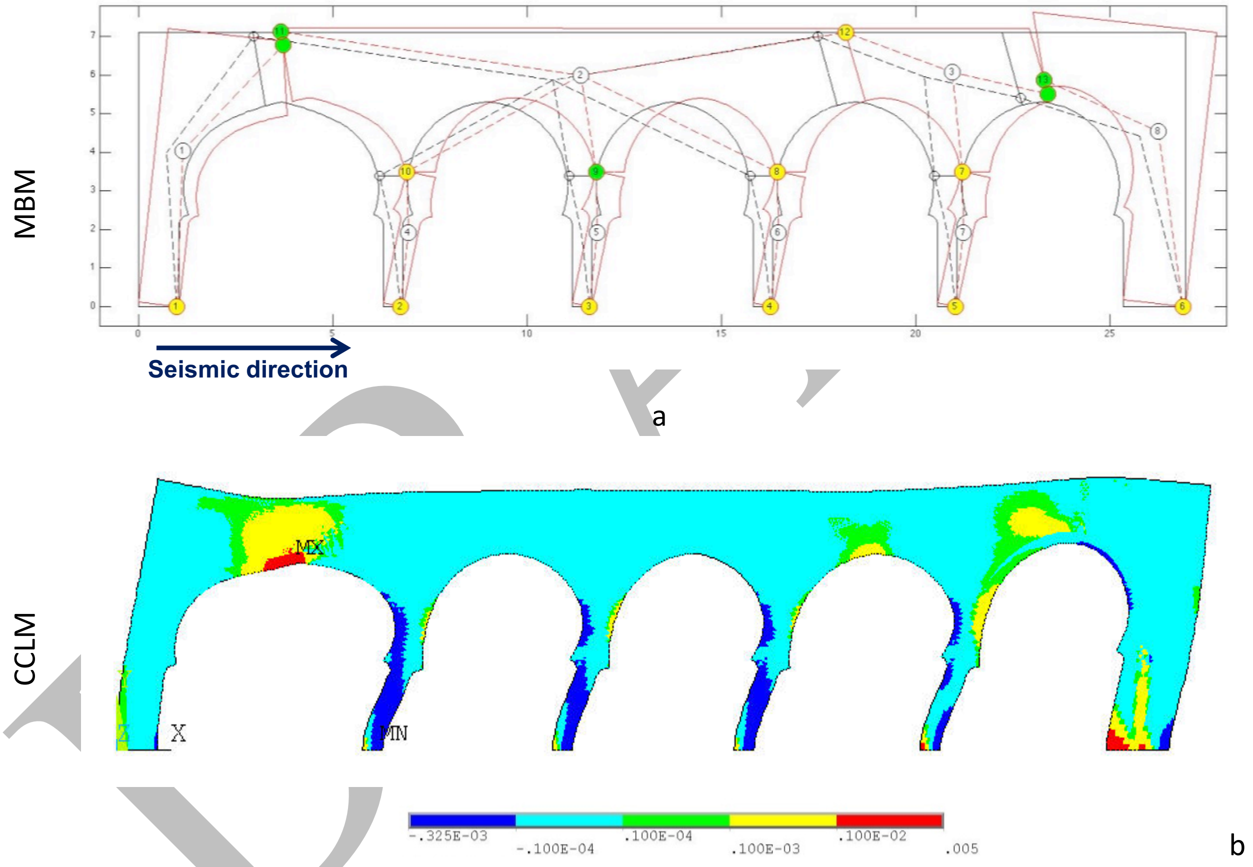


Figure 4.128. The collapse mechanism in the MBM (a) and the localization of damage in the CCLM (b) of a generic arcade system.

Another modeling approach that can be used is the Discrete Interfaces Model (DIM), that can be useful to investigate more complex assets and damage state or for a fully dynamic analysis with large displacements. In this case the model is able to predict by the analysis the evolution of the masonry portions (blocks) really involved in failure mode.

3.5.4.2 Considerations on modelling 3D mechanisms

Draft

This kind of response rises because vertical structures are not able to sustain vault horizontal thrust or because they are subject to different horizontal displacements (interstorey drift ratios), which induce shear distortions in the horizontal plane. A MBM approach is not able to simulate the complex damage modes that can affect this kind of assets and it is thus advisable to use CCLM or DIM strategies, in order to capture also a 3D involved mechanism.

A typical example of asset that tends to have this kind of response is the cloister (see Figure 4.129) where frequently among the vertical structures that support the vaults there is a different stiffness (a typical condition is that of a wall on one side and pillars or columns on the other).

Figure 4.130 shows another example, that of a masonry arch bridge made by a set of arcades in which the extremes have to be considered clamped to the soil. If we consider the seismic action in longitudinal direction (x), the system may be studied through the MBM approach considering the development of the possible activated kinematism. However, if the seismic force acts along the transversal direction (y), the response of the system involves the horizontal plane of the vault and can activate also 3-dimensional mechanisms that cannot be simulated through the MBM approach. The only way to model the structure and their seismic response is by using a CCLM or a DIM approach.

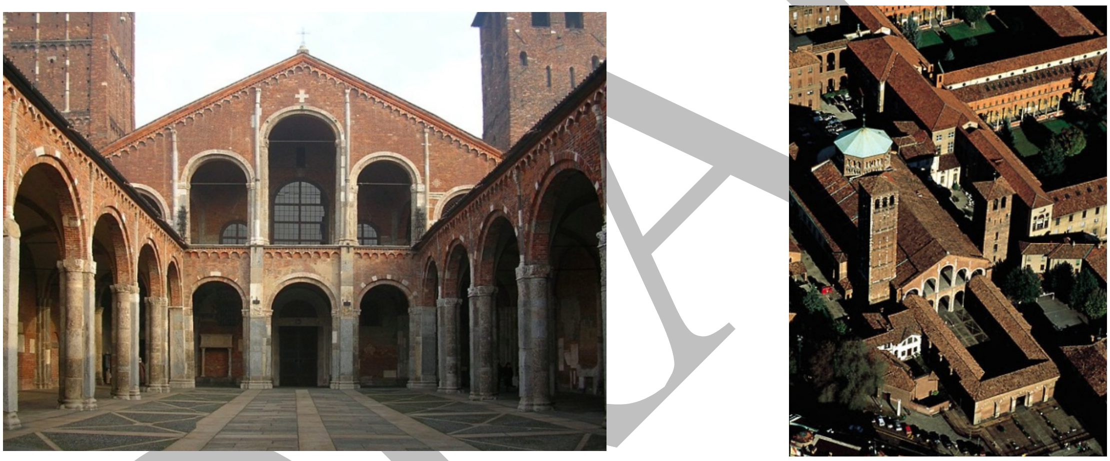


Figure 4.129. Cloister in the forward part of Sant’Ambrogio Church (Milan, Italy)

DRAFT

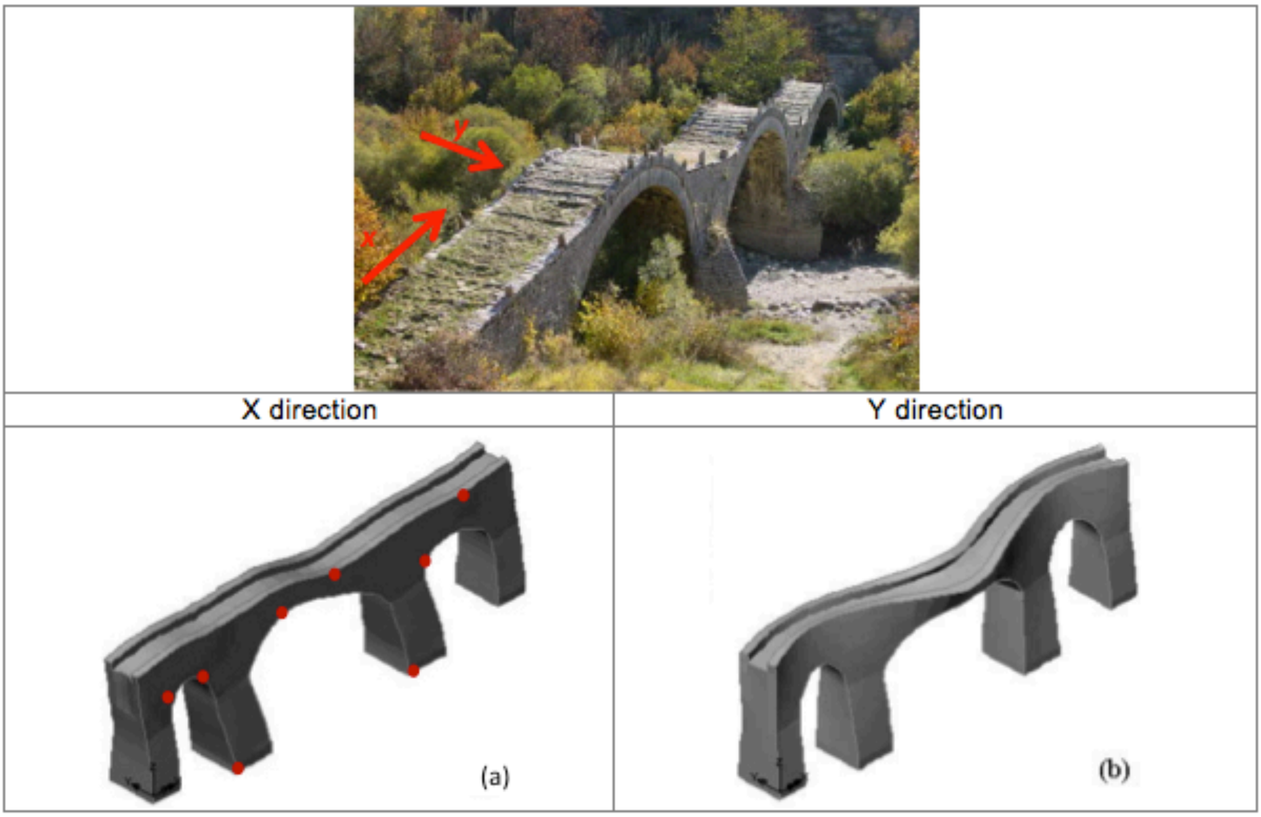


Figure 4.130. Seismic response of a bridge considering two different directions.

3.5.5 *Class E (Massive structures)*

This class of assets includes fortresses, defensive city walls and, in general, all masonry structures made of massive walls, so that their thickness have a 3-dimensional behaviour different from that of standard thinner walls. Typically, these assets suffer local damage such as cracks or spalling due to soil or internal core thrust, detachment of external masonry leaf, partial collapses.

The choice of the most suitable models should be made through a “case-by-case” approach. In general, massive structures are not vulnerable to global collapse mechanisms, as due to their dimensions they would have great displacement capacities, and their damage modes are related to local mechanisms that generally involve, for example, slender elements of connection among bigger blocks.

In the case of thick multi-leaf walls, which generally are characterized by a poor connection between leaves, the assessment of the seismic vulnerability of the external leaf can be performed by means of the MBM analysis, modeling the material of the internal leaf, usually loose and incoherent, as a mass that generates an internal pressure and that gives a contribution to activate the overturning mechanism (Figure 4.131). This modeling approach is obviously based on the assumption that the location of cracks in the external masonry leaf is predefined.

A more accurate evaluation can be made by DIM, which is able to evaluate the critical joints and the relative displacements among blocks (Figure 4.132).

DRAFT

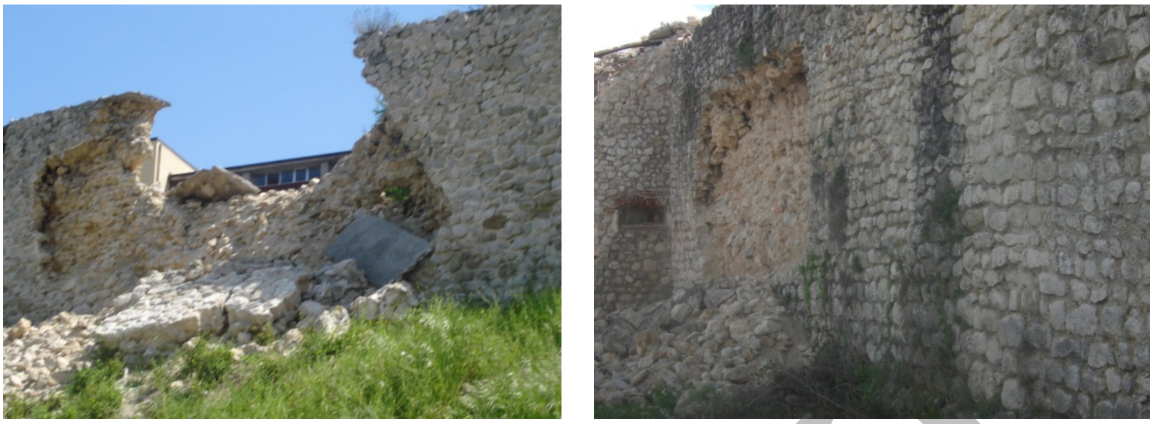
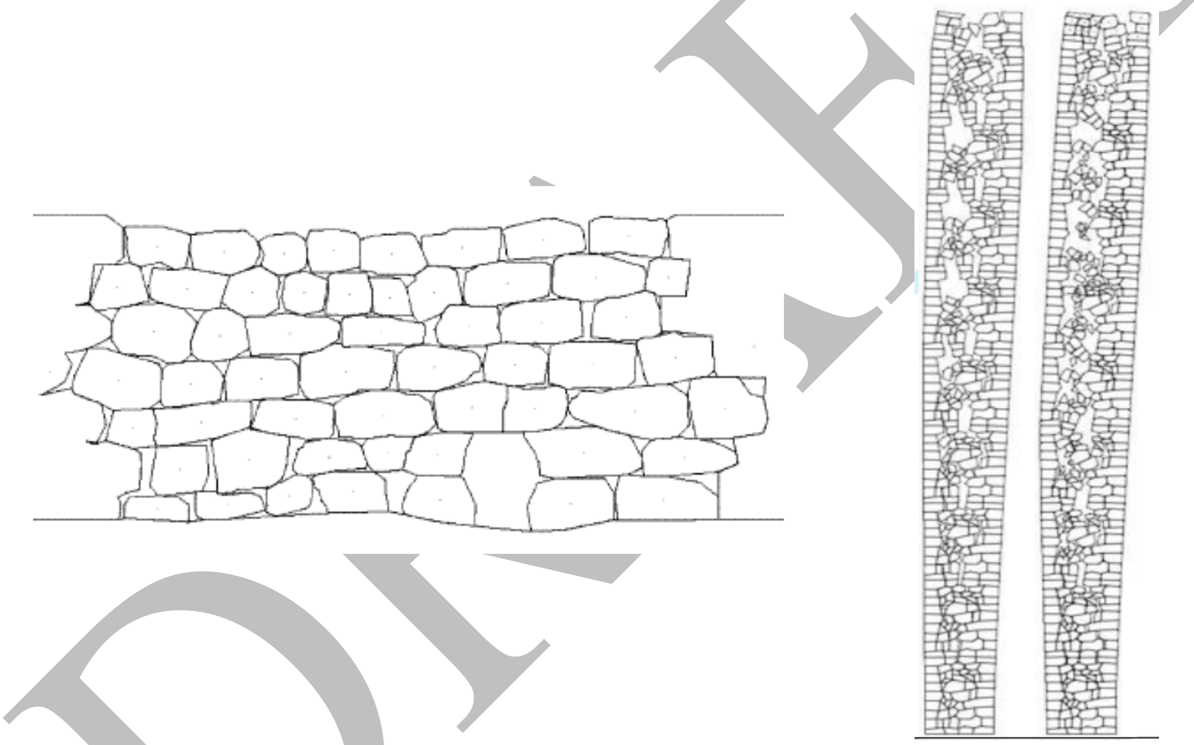


Figure 4.131. Partial collapse of the external leaf of the defensive masonry wall of L'Aquila (2009 earthquake).



(a) (b)

Figure 4.132. Examples of irregular dry stone external leaf modeling using discrete elements (Roberti and Spina, 2001(a) and De Felice, 2011 (b))

3.5.6 Class F (Dry blocks structures)

Assets belonging to *class F* are masonry structures made of very large blocks (size comparable with the dimension of the whole structure) that are usually subjected to overturning and loss of stability under seismic actions. It includes also archaeological ruins made of self-standing elements that may be subjected to overturning due to their slenderness.

DRAFT

The seismic performance assessment of these assets commonly refers to two different type of responses related to two different systems: a) single block, or multi blocks system whose main response can be assumed equal to that of a single one; b) multi blocks systems.

3.5.6.1 Single Block dynamic response

Single blocks or multi-blocks systems whose response can be assumed equal to that of a single one generally can be modelled by means of MBM approach. It is important to note that the possibility to model the multi blocks systems as a single one is due to the fact that in some cases the development of sliding between blocks is prevented by pins or notches in the stone (the friction usually is not sufficient to prevent from sliding in multi-drums columns). However, even if sliding is prevented, a single block approach is admissible if the number of blocks is not too high.

MBM can be used to perform both static non-linear and dynamic non-linear analyses. In the static non-linear field, it may be considered only a prevalent mechanism of response of the structure respect to a well-defined distribution of forces, in order to evaluate the ultimate displacement capacity of the system.

3.5.6.2 Multi Blocks systems

Classical monuments and in particular columns are made by carefully fitted stones (drums in the case of columns), which are juxtaposed without mortar. During a strong earthquake, each single stone can slide and/or rock independently or together with the adjacent ones (Figure 4.133). Their response is complex, highly nonlinear and it is practically impossible to treat the problem analytically. In order to overcome this difficulty, the numerical calculation of the dynamic response of these structures by means of DIM approach seems to be an efficient tool.

DRAFT

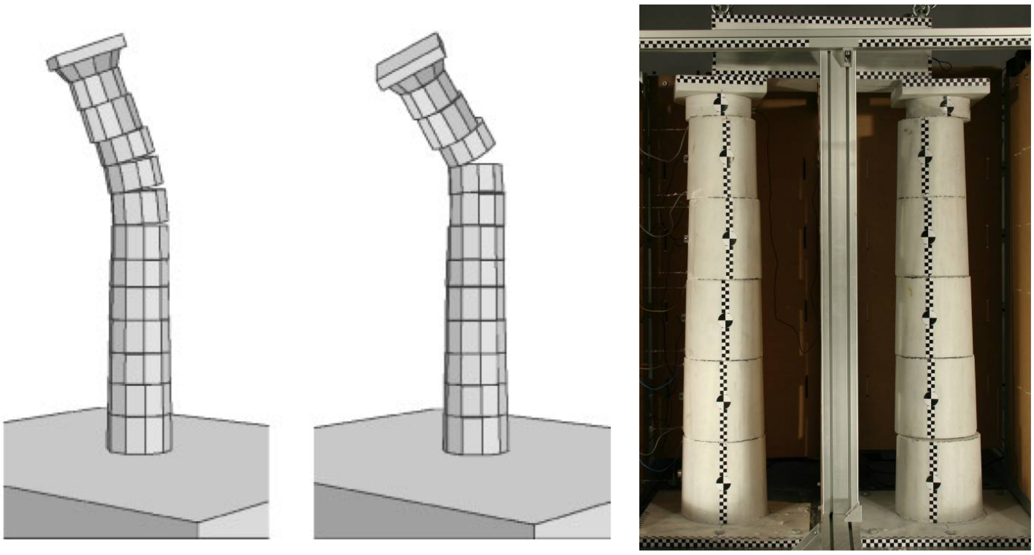


Figure 4.133. Failure process of single-column model (Psycharis et al. 2003)

3.6 Use of the models for the seismic assessment of heritage buildings

It is evident that the modelling strategies illustrated in the previous paragraphs could be applied for several and general purposes, as an example for describing the construction behaviour in serviceability conditions or at ultimate limit state, as well as for interpreting the structural health monitoring and designing strengthening interventions, etc…In the following, the attention is focused on their use for the seismic analysis.

Finally, it seems useful remarking how the models described in the previous paragraphs may be used to perform different types of analysis (static or dynamics, linear or non-linear…). With reference to the performance seismic assessment, modern national and international norms (as an example Eurocode 8 - EC8, FEMA 306, NTC2018) consider four main types of analyses: linear static analysis, linear dynamic (typically multimodal with response spectrum), non-linear static and non-linear dynamic. Moreover, with specific reference to the analysis methods for local mechanisms, the Italian Code (NTC2018) propose the adoption of a kinematic linear and non-linear approach (basically founded on the Equilibrium Limit Analysis which, in case of the non-linear approach, is applied on a set of varied displacement configurations in order to obtain a proper capacity curve).

As already discussed, the modern seismic norms for the design of new buildings, as well as the most advanced recommendations for the evaluation and rehabilitation of the existing ones, are based on the Performance Based Assessment (PBA).

Among the different analysis methods, PBA generally refers to non-linear static analyses and capacity spectrum method. According to this approach, the response of a building subjected to increasing horizontal forces/displacements is evaluated by an incremental static non-linear analysis; the response is then represented in terms of a force-displacement curve ("pushover" curve) able to describe the overall inelastic response of the structure in terms of stiffness, overall strength and ultimate displacement capacity. Once defined the pushover curve, it is possible to associate proper building damage levels (usually four, that is 1-slight damage, 2-moderate damage, 3-heavy damage, 4-severe damage) to fixed displacement values. Performance levels should be defined in relation to different performance targets, linked to functionality and cultural value of buildings (both structural damage and response of non-structural elements and artistic assets shall be considered).

The non-linear static procedure require: firstly, the conversion of the original multiple degree of freedom structure (NDOF), represented by the pushover curve, to an equivalent single degree of freedom structure (SDOF); secondly, the comparison between the capacity of the equivalent SDOF system and the demand expected, described by an elastic spectrum appropriately reduced (according to one of the two approaches proposed in literature, that is inelastic or overdamped spectra), in order to identify the *performance point* (PP) of the structure.

In the context of PBA, the application of linear analysis methods is less suitable than non-linear ones, since non-linear response resources of structure may be taken into account only in an approximate way through *behaviour factors* which are evaluated conventionally for each typology class (indeed for some classes of historic buildings, like as towers and bridges, specific values lack almost at all). Moreover, the definition of different target levels results quite arbitrary and graduated only to the use of different values of the *behaviour factor*: As an example, in the case of *immediate occupancy* level, reference should be made to the over-strength quota of the behaviour factor; in case of *life safety* level, the full *behaviour factor* should be

DRAFT

considered; in case of *collapse prevention* level, its value should be further increased (but specific recommendations lack in norms).

Although non-linear static analyses are commonly adopted in PBA (due to both acceptable computational efforts and consolidated verification procedures), also non-linear dynamic analyses may represent an effective tool. On the one hand, the results provided by non-linear dynamic analysis represent a reliable tool to validate non-linear static procedures; on the other hand, as specified in detail in the following, non-linear dynamic analyses may help to investigate the role of additional parameters characterizing the seismic input in the actual response of different structures.

Finally, it seems important noting that PBA and non-linear static procedure concepts may be applied to both global response and local mechanisms. In fact, although initially non-linear static procedures have been developed focusing the attention on the global response of buildings, recently they have been applied also to local mechanisms. In particular, reference is made to the Italian Code - NTC, which propose a methodology to obtain the capacity curve by applying the Equilibrium Limit Analysis to a set of varied displacement configurations of a selected mechanism (*Non Linear Kinematic Approach*).

In the following, some specific issues related to the use of the different types of model in the context of PBA are discussed. In particular, the attention will be focused on: issues generically related to the application of non-linear static procedures to historic buildings; issues specifically related to the definition of limit states on the basis of the results provided by models; additional issues related to the use of non-linear dynamic analyses.

Issues generically related to the application of non-linear static procedures to historic buildings

Although the use of seismic verification procedures according to the Capacity Spectrum Method, is well established in the literature and norms, their application to masonry historic buildings still presents some open issues. This is due to specific features (e.g. the presence of flexible floors) which may characterize them with respect to other structural typologies (that is steel and reinforced concrete ones) which these procedures have been originally developed to.

Concerning global response, the application of non-linear static analyses to define the capacity curve poses the following main critical issues: (i) choice of the control node; (ii) definition of the load pattern.

Regarding the control node (i), issues related to its proper choice both in elevation and plan may be highlighted. Regarding the elevation, it is necessary that the control node is selected upon the level in which the collapse occurs, in order to obtain theoretically consistent results. For this reasons, norms commonly propose to assume the control node at the top floor. Regarding the in plan location, the choice represents a very crucial issue in case of existing buildings with wooden floors or vaults. In fact, while in the case of rigid floors the results are almost insensitive to the position of the control node, in the case of flexible ones they strongly depend on it, in particular in presence of shear masonry walls characterized by very different stiffness. In these latter case, the global displacement capacity is greater and the numerical results are more accurate if the control node is selected in a collapsing wall rather than in one remaining in the elastic range. As a consequence, it should be correct to locate the control node in that wall which firstly fails. A reasonable compromise is to assume, for the analysis, a generic node at the level of the upper floor, but to refer to the

DRAFT

average displacement of all nodes located at this level (eventually weighted with the pushover nodal force) for the pushover curve.

Regarding the load pattern (ii), norms usually propose to assume the following two patterns: a) *uniform* (proportional to nodal mass); b) *modal* (proportional to a lateral force distribution consistent with the first fundamental modal shape of the structure).

The *modal* pattern aims to describe the seismic action by emphasizing the dynamic amplification provided by the structure. However, it could be not reliable in the case of flexible floors, in which a very low fraction of the mass may participate to the fundamental mode (the limit case is that in which each wall is independent from the others). In this case, it could be preferable referring to a loading pattern proportional to the product of the nodal masses and the nodal heights (quoted as “triangular”): in this way, all the structural masses would be involved in the pushover analysis.

The two load patterns considered (*modal* and *triangular*) could not be adequate in the case of structures with irregularity in elevation (as an example the case of an original masonry building with raising-up by mean of a r.c. frame). In these cases, it seems reasonable referring to the *adaptive pushover* analysis or the *multi-modal* one. According to the *adaptive pushover*, at each step of the analysis the load pattern is upgraded as a function of the evolution of the non-linear response occurred in the structure. However, it is worth noting that, whereas for other structural types (such as r.c. or steel buildings) the application of this procedure has been widely investigated, in case of masonry structures very few references may be found in the literature (Galasco et al. 2006). Further researches are needed in this field, due to the distinctive features of masonry structures (such as the softening response of masonry under shear and the presence of flexible floors).

Concerning local mechanisms, with particular reference to Non Linear Kinematic Approach, it seems important remarking the importance to analyse the shape the capacity curve as function of the mechanism considered. In fact, varying the position of hinges, both the collapse multiplier and the displacement capacity may vary (and in general not in the same direction, that it a decreasing of multiplier should be associated to a displacement increasing): thus it quite difficult establish at priori what is the worse condition in PBA.

Issues specifically related to the definition of limit states starting from results provided by models

The definition of limit states on the capacity curve proposed in literature and norms is based on the following two approaches: a *heuristic* approach (i.e.: damage level 1 is attained when the force is 70% of the maximum strength; damage level 2 when the building reach its maximum strength; damage 3 when the force decreases of 20% with respect to the maximum strength; …); a *structural element* approach (e.g. as proposed in ASCE/SEI 41-06, the limit state on the capacity curve is fixed depending on the attainment of the limit state in one structural element). It seems important noting that recommendations proposed in norms refer only to the global response and only in few cases (NTC) criteria are proposed for local response also.

The evaluation of the seismic response (and the consequent definition of proper limit states) at *structural element* and *macroelement* scale passes through the definition of proper damage variables. As an example, with particular reference to the *structural element* behaviour, damage can be defined as a function of well-defined displacements, rotations or deformations (depending on the type of structural element and building).

DRAFT

The evaluation of the damage variables represents thus a fundamental issue in the use of the models. Considering *continuous models*, it seems worth noting how some differences can be found between CCLM and SEM. As an example, with particular reference to the case of the in-plane response of masonry walls, the attainment of such limit states is in general associated with the attainment of predefined drift values of the pier and spandrel elements (the drift is in this case the damage variable adopted). Since in CCLM the structure is modelled as a continuum, the identification of the elements on which this parameter should be monitored might be ambiguous (differently from the case of SEM models in which this choice is made at priori). The ambiguity derives from the need to conventionally identify pier and spandrel elements within continuum, on the basis of non-linear response occurred. Moreover, once panels and sections to be monitored have been identified, the evaluation of the parameter may imply repeated average operations performed ex-post: as an example, since in general each section is characterized by several nodes, average operations are required to define single reference values.

With reference to the use of non-linear dynamic analyses, the use of *incremental dynamic analysis* (IDA) seems particularly attractive. In particular, IDA involves subjecting a structural model to one (or more) ground motion record(s), each scaled to multiple levels in order to describe the full range of structural behaviour, from elasticity to collapse (as an alternative to scale the same accelerogram – operation which poses some critical points from a theoretical point of view - several accelerograms with different PGA values and other parameters of the input demand may be used). The result of IDA is usually represented in terms of a curve that shows an Engineering Demand Parameter (EDP) plotted against an Intensity Measure (IM) used to control the increment of the ground motion. Also in this case, similarly to non-linear static analyses, a capacity curve may be obtained by plotting, for each dynamic analysis performed, the maximum shear base force reached and the maximum displacement of a control node. Different EDP may be chosen in order to transpose the achieved results in terms of limit states: interstorey drift value, maximum displacement, significant variation in the energy dissipated.... In particular, it would be very interesting to analyse the possibility to adopt damage indexes similar to those proposed in the literature for other structural typologies (as an example, the Park and Ang index proposed in the case of reinforced concrete structures in Park and Ang 1985). Moreover, by adopting different IM aimed to describe the input demand (PGA, PGV, Arias Intensity, Housner's Intensity, Spectral displacement at the period of different modes, etc...) it could be assessed what parameters are the most suitable to describe the progressing of non-linear seismic response. With reference to this, as an example, some authors (Apostolou et al. 2007), by performing non-linear dynamic analyses, stressed how overturning phenomena in rigid blocks (as an example the case of single columns) are not so strictly related to PGA values (it means that a block, subjected to two different records characterized by different PGA values, not necessarily collapses in correspondence of the higher value) or other intensity measures (Ishiyama 1982, Sorrentino and Masiani 2007).

Additional issues specifically related to the use of non-linear dynamic analyses

The comparison between results carried out by non-linear step-by-step dynamic analyses and non-linear static ones will represent a fundamental tool to validate and improve the use of non-linear static procedures (which of course are less expensive in terms of computational effort and more applicable from an engineering and practice point of view). As an example, the use of IDA approach and the analysis of sensitivity of results with respect to several intensity measures, could lead to an improvement also of non-linear static procedures, which basically refer to the use of response spectra to describe the seismic input. Moreover,

DRAFT

with specific reference to the different approaches proposed in the literature to reduce elastic spectra adopted in non-linear static procedures (whether *inelastic* or *overdamped*), the comparison with non-linear step-by-step dynamic analyses could allow to define what approach is the most suitable varying the typological class examined. In addition, in case of use of *overdamped* spectra, a proper definition of the equivalent damping associated to different displacement values for each building type and macroelement is still needed.

Finally, in case of local mechanisms, another important point is the evaluation of the seismic demand for local mechanisms at the highest storeys taking into account the amplification of motion due to their position in the main building. Also in this case, the results achieved by performing non-linear dynamic analyses could represent a precious resource.

DRAFT

4. Interventions techniques

4.1 Strategies for the selection of interventions

Rehabilitation decisions and selection of interventions must fulfil with the goals of safety and durability, and should be limited in order to produce minimum impact on the historical building. It is useful to remember that periodic maintenance is the starting action to avoid material deterioration, prevent from the need of later major transformative interventions and may also help in the seismic preservation.

The selection of intervention techniques should be evaluated on a case-by-case basis, preferring the least invasive ones and those with the greatest compatibility to conservation criteria, taking into account the requirements of safety and durability. Precedence should be given to reversible interventions (as much as possible); new materials can be useful, if they are compatible with the original ones and do not affect their durability.

As much as possible, interventions should respect the original constructive techniques and structural concepts of the construction, as well as any significant transformations that may have occurred throughout the history of the building. From this point of view, the damaged structural elements should be repaired rather than substituted, whenever possible, and the deformations and alterations which testify its past should be maintained, eventually adopting measures to limit the negative effects on safety conditions.

Particular attention should be paid to the executive phase of the interventions, in order to check the accuracy of the hypothesis assumed in the design process and insure their real effectiveness. All realized works must be documented in detail, in order to be available for future generations, when further interventions will be necessary.

Sometimes historical buildings are subjected to interventions related to the update of technological systems: electrical, water and sanitary, heating, air conditioning, fire. Even if they are not considered structural intervention, they can be invasive and reduce the capacity of structural elements. A PBA is necessary when the impact on the construction is significant.

An intervention must also be evaluated considering its cost and comparing it to the benefits produced and the actual need.

In the following, some general indications for the choice of seismic strengthening interventions for masonry buildings are given, with reference to techniques widely used today. For each kind of pathology or vulnerability more than one intervention is possible, with distinct characteristics in terms of effectiveness, invasiveness, reversibility, compatibility, durability and cost.

The possibility of adopting provisional interventions (such as shorings) should not be overlooked, which due to their intrinsic reversibility appear interesting with respect to preservation and after careful evaluation, may also result to be the definitive solution for historic buildings.

The following indications must not be seen as a list of interventions to perform anywhere and everywhere, but only as possible solutions to adopt in cases where they are really needed, according to the outcome of the PBA. Obviously, techniques that have not been cited herein should not be excluded; in particular,

DRAFT

innovative methods are promising, even if their effectiveness and impact on the cultural asset have to be carefully evaluated in advance.

4.2 Interventions to improve connections

Such interventions are aimed to provide a global behaviour to the building, through the realisation of good clamping between walls and effective floor to wall connections. Moreover, eventual thrusts due to arched structures or roof rafters must be verified, making sure the connections are sufficient to balance them. The realisation of these interventions is an essential prerequisite for applying global seismic analysis methods to the building, which are based on the in-plane behaviour of the masonry walls, assuming their stability with respect to out-of-plane seismic actions.

The insertion of tie-rods, whether in steel or other materials, placed in correspondence to bearing walls at the level of floor diaphragms, can aid the behaviour of the whole building, since it confers good connection between orthogonal walls and supply an effective constraint against out-of-plane overturning of the masonry panels, when stability is not guaranteed by floors or other structures. Moreover, the insertion of tie-rods improves in-plane behaviour of walls with openings, because it increases the seismic capacity of masonry spandrels. Simple bolts or slabs may be utilised to anchor tie-rods to walls; with the exception of particularly poor masonry walls, made up of small sized elements, the use of bolts to plates is preferable due to the fact they bear a larger wall area. In any case, the dimensions of the end-constraints must be chosen on the basis of the characteristics of the masonry elements (panels, columns, pillars) to which they are connected. Often local consolidation of masonry is necessary in the anchoring zone. It is not advisable to place the end-constraints into the wall thickness, especially for multi-leaf masonry. Utilising anchoring for adherence (through injections of resin or cement-based mortar into the masonry) should be carefully considered, due to both possible incompatibility of the materials as well as the difficulty of controlling the effectiveness of the anchoring itself. The tie-rods must generally be positioned next to principal load bearing walls; if it is necessary to core longitudinally the wall, tie-rods should not be injected, in order to render the intervention reversible and allow an eventual re-tensioning. As far as the tensioning of the rods is concerned, limited tension should be used in order to induce in masonry limited compressive stresses, well below strength values.

External ties with metallic or composite materials can guarantee an effective connection between orthogonal walls when dealing with smaller buildings, where the length of the sides of the ties is not too high, or when additional anchoring are provided in correspondence with internal intermediate masonry walls. It is necessary to avoid the concentrated insurgence of tension near sharp masonry corners, for example with opportune elements of diffusion; when strips made of composite materials are used, the angles must be smoothed.

Clamping between adjacent parts or intersecting walls (*cuci-scuci* technique) can be used (with stone or brick elements) whenever the junctions between masonry elements are deteriorated (due to cracking) or particularly decayed. These interventions are destructive and substitutive of ancient materials by their very nature, and therefore should be used with caution, on a case-by-case basis.

The use of reinforced bars perforations must be limited to cases where there are no other feasible solutions due to the high degree of invasiveness of such elements and their dubious effectiveness, especially in the

DRAFT

presence of multi-leaf masonry panels. The durability of the inserted bars must be guaranteed (stainless steel, composite materials, etc.) as well as the compatibility of the injected cement.

It should be noted that these local junctures are effective for the overall behaviour of the building only when good masonry walls are present. When masonry is of poor quality, the use of tie-rods is preferred as it guarantees a better connection.

Stringcourses (or tie-beams) at roof level can provide an effective solution for joining walls in zones where the masonry is less cohesive due to limited levels of vertical compression, and to improve the connection with the roof coverings. They can be realised in the following ways:

- In reinforced masonry, which permits connection through a technique that provides maximum preservation of the characteristics of the existing masonry. In fact, they must be realised with masonry of good characteristics, usually with solid bricks and hydraulic mortar; in a small inner core, steel or composite bars are placed and made adherent to the masonry by concrete. The connection between the tie-beam and the underlying masonry wall usually is simply guaranteed by a good adherence, the friction and the irregularity of the contact surface (in some cases it may be advisable to consolidate the tops part of the masonry wall by mortar injections). The use of inclined reinforced bar perforations should be avoided whenever possible.

- In steel, which represents a valid alternative due to its lightness and limited invasiveness. They can be realized in two ways:

a) by way of a reticular truss structure, placed horizontally on the top surface of the masonry wall, made of steel angular profiles or plates which are welded together and connected by way of pseudo-vertical reinforced bars perforations;
b) by way of plates or profiles at the two surfaces of masonry wall, placed just beneath the top and connected by pass-through bars. In the presence of poor quality masonry, this intervention must be joined with consolidation mortar injections to improve the masonry portions involved. Metallic stringcourses are also particularly suitable for connecting wooden rafters of the roof and contribute to the balance of eventual thrusts.

- In reinforced concrete (r.c.), only if the height of the tie-beam is limited, to avoid excessive weight and stiffness, which has proven to be negative due to the generation of shear stresses between r.c. and masonry, with consequent sliding and crumbling of the latter. In particular, such effects are manifested when the roof covering has also been stiffened and weighed down by a r.c. slab down. In general, it is advisable to consolidate the tops of the masonry in order to limit the different stiffness of the two elements. The connection with reinforced perforations may be adopted only when considered really necessary and after having verified that bars can be anchored efficaciously to the eventually consolidated masonry.

The effectiveness of floor-to-wall and roof-to-wall connection is necessary to avoid beams pull-out, which causes ceiling collapse, and contribute to the box-type behaviour. Moreover, these connections permit the horizontal diaphragms to better distribute seismic actions among vertical elements. In the case of intermediate diaphragms (at floor levels), the ends of the wooden beams can be anchored to the masonry through metallic elements or other materials that are resistant to traction, anchored to the opposite wall surfaces.

DRAFT

The insertion of stringcourses in r.c. into the thickness of the walls at the intermediate levels has a negative impact on the seismic behaviour of the walls, besides of being incompatible with conservation criteria. Eventually, in the case of walls that are very flexible out-of-plane, due to the high distance between orthogonal bearing walls, steel plates stringcourses may prove useful; they are made with plates or profiles on two surfaces, connected by way of pass-through bars. They furnish a significant flexional out-of-plane stiffness to the walls and prevent from the development of cracking mechanisms to the masonry spandrels. In the case of external walls, the effectiveness of such stringcourses with only one internal profile, anchored to the external masonry leaf by way of passive anchored bar must be verified carefully.

4.3 Interventions to reduce thrusts of masonry vaults and their strengthening

Strengthening interventions to arched or vaulted structures may be realised by utilising the traditional technique of tie-rods, which balance the thrusts induced on the vertical walls and prevent from the movement of the springs. The tie-rods are normally placed at the level of haunches; whenever this position is not possible, the tie-rods can be connected at diverse levels (e.g. the extrados) as long as the shear and bending stresses induced to the wall has been verified. The tie-rods must be put in place with adequate pre-solicitation to absorb part of the thrust action, estimated by way of calculations (an excessive traction may cause localised damage).

In order to balance the thrusts of vaulted structures, buttresses or masonry wall thickening can be considered. These interventions have an appearance impact on the construction and effectiveness is subject to the creation of good clamping of these added elements to the existing masonry wall; to this end, discrete connections with stone or brick elements can be placed and it is also necessary to create an adequate foundation.

The extrados realisation of a concrete cap, whether reinforced or not, should be avoided for the reduction of the state of compression in the masonry vault and the increase in seismic masses, besides of the loss in terms of cultural value in the historic building.

It is possible to apply at the extrados stripes of composite materials (FRP) because it is lighter and also removable technique of intervention. Nevertheless, the following issues should be considered: a) differences in transpiration that may occur in the areas where FRP are glued (critical in case of presence of frescoes); b) durability (the experience of the behaviour over time, both of the fibres as well as the resin used as glues, is still rather limited); c) incomplete reversibility (the superficial parts of the masonry remain impregnated with resin). The position of the FRP stripes, especially in the presence of complex vaults, should be defined on the basis of an accurate structural analysis that demonstrates its effectiveness. The application of FRP stripes at the intrados is effective only when coupled with the realisation of sub-arches, made of masonry, steel or other materials (e.g. in-situ glued laminated timber), which are able to balance the peeling phenomenon. As an alternative, through-thickness anchoring regularly spaced along the intrados can be used, but this solution is nevertheless very invasive.

In order to reduce thrusts, it is possible to intervene by reducing the extrados loads (filling materials can be lightened), but it is necessary to pay attention to the alteration of the original thrust line in the arch. Moreover, it is worth noting that permanent loads render the vault less sensitive to accidental loads. To this end, the use of a light conglomerate can be an effective solution, because it contributes to prevent from the

DRAFT

formation of collapse mechanisms; however, the mechanical properties of this filling material must be very poor, in order to assure reversibility and allow future interventions.

Obviously, in the presence of cracks, repair must be made in order to restore the original contact between the ashlars (or in mortar joints) by way of local mortar injections, that can be realized both from the extrados and the intrados. In particular cases, wedges can be utilised, in order to force the contact between disconnected elements.

Vault portions in which crushing of masonry occurred must be substituted. Particular attention must be paid in cases where significant loss of the original curvature of the arch or vault is present. Its recovery is often problematic, thus a sub-arch or other integrative structures can be adopted. An interesting alternative solution is the positioning at the extrados of not-injected cables, which after a post-tensioning transfer radial actions which modify the trust line so that it was closer to the middle line of the arch; this intervention is also able to recover a little bit the original curvature.

4.4 Interventions to reduce the flexibility of horizontal diaphragms

The floor diaphragms must be effectively connected to the masonry walls by means of a sufficient support length and connection elements that impede the pull-out of timber beams or steel profiles. The role of diaphragms in seismic behaviour of masonry buildings is to transfer lateral actions to the walls which are parallel to the direction of the earthquake. Moreover, they improve the constraints for the walls loaded out-of-plane. To this end, it is usually sufficient a limited in-plane stiffening of the horizontal diaphragms (as can be demonstrated by equivalent frame modelling of buildings), that must be realized without increasing the dead loads. Exaggerated stiffening, besides of the increase of seismic masses, tends to concentrate forces into few stiffer walls, usually limiting the displacement capacity at failure; moreover, in case of plan irregularity, torsional effects are increased and the exterior walls turn out to be overloaded. Compatible to the above goals, it is opportune that wooden diaphragms are preserved as much as possible, due to their lightness, with simple intervention.

A limited stiffening of wooden floor diaphragms can be performed at the extrados on the planks. One can set a second wooden plank over an existing one, which should be posed in an orthogonal or slanted direction and by paying particular attention to junctions with lateral walls. An alternative or addition is to use diagonal reinforcements with steel plates or composite material stripes (FRP), which are fixed to the original plank. An analogous benefit can be obtained by diagonal bracing made of steel bars of cables. In the case of ordinary wooden floors, the junction between two parallel walls to the beams should be done carefully, for example by putting fixed bands to the planking and anchoring them to the masonry.

When strengthening of floor diaphragms is necessary for static vertical actions, with the wood-wood technique it is possible to limit the flexional deformability and increase resistance with a second layer of planking, placed orthogonally to the existing planking. The new continuous planks are connected to the beams by way of pins (even in timber).

Another reinforcement technique is the realization over the plank of a thin reinforced concrete slab (eventually with a lightened material), collaborating with the original timber beams through proper connectors. The effects of this type of intervention should be evaluated in relation to specific conservation requirements.

DRAFT

When the wooden elements are not adequately connected to the masonry, it is necessary to link the floor diaphragm to the walls by way of regularly placed elements.

In cases where diaphragms with steel profiles are present, with interposed arched solid bricks or hollow-clay tiles, it may be necessary to connect them by way of transverse metallic bands, welded to either the intrados or the extrados.

4.5 Interventions on the roof covering structure

The original wooden roof has to be maintained, not only because of the conservation of its cultural value but also due to its flexibility, which is compatible with that of the system of orthogonal masonry walls, and its lightness, which limits the seismic actions just in the highest parts of the building.

Usually, links and connections among timber elements should be improved, as well as those with the top of the masonry wall; technological details should be compatible and similar to the original historical constructive details, when effective. The ends of timber rafters and king trusses can be connected to steel plates anchored to the walls; if a stringcourse (tie-beam) is present, these connections are very easy to be realized.

Whenever roofs produce thrusts to the perimeter masonry walls, if possible it is better to implement the structural configuration of the original roof in order to eliminate these thrusts, by adding steel and/or timber elements. Otherwise, a tie-beam stiff enough must be added on the top of the wall, in order to balance these thrusts.

In the cases of timber trusses, good connections in joints must be present, which are necessary for avoiding shifting and breakage during the seismic action. This can be improved by plates and metal bars, or with other materials (e.g. FRP).

If the roof is not stiff enough in the sloped diaphragms, some improvement may be useful, on condition that the total weight is not increased and the structure remains not totally rigid. Technical solutions are similar to those introduced in case of horizontal timber floors (double timber planks, FRP, diagonal steel plates). A proper system of bracing, by crossed post-tensioned bars, can be introduced at the intrados.

4.6 Interventions for increasing the strength of masonry panels

These interventions are aimed at both repairing of deteriorated and damaged masonry and improving the mechanical properties of the masonry. The technical solutions applied should be evaluated also on the basis of the typology and the quality of the masonry. The interventions must utilise materials with physical-chemical and mechanical characteristics analogous with or at the very least compatible with the building existing materials. The interventions should aim to provide a substantially uniform resistance and stiffness to the walls, which can also be accomplished by the improvement of clamping between walls, whenever needed. The insertion of materials diverse from the existing masonry, and particularly cement-base mortars, should be utilised with caution and only where the cost-benefit ratio (with reference to safety and

DRAFT

conservation) is favourable (e.g. when the negative impact of cement is minor than the loss connected to the need of demolishing and replacing elements).

Depending on the case one can:

- Repair at local level cracks or deteriorated masonry portions;
- Reconstruct masonry portions in correspondence to recesses, unused chimney flues, badly closed openings;
- Improve the characteristics of the particularly poor masonry types, which can be inadequate due to quality of mortar or size and assembling of blocks.

Cuci-Scuci intervention consists of disassembling masonry blocks in the deteriorated portions (e.g. along a crack) and reassembling masonry again, with the aim of bringing back continuity. The use of materials that are similar to the original ones in shape, size, stiffness and resistance is advised. The new elements should be connected to the existing masonry with adequate clamping in the external leaves and, when possible, in the transverse section, in order to provide maximum homogeneity and monolithic nature to the repaired wall. Such interventions can also be utilised for closing recesses and unused chimney flues, particularly when they are positioned near corners or at the intersection between masonry walls.

The use of mortar injections is aimed at improving the mechanical characteristics of masonry that needs strengthening. This technique cannot provide adequate clamping between the walls. Such an intervention proves to be ineffective if employed on types of walls that by their very nature are not easy to be injected (scarce presence of voids and lack of interconnections among them). Particular attention should be paid to the choice of the injecting pressure of the mix, in order to avoid the insurgence of transverse dilatation. Particular care should be given to the choice of the mixture to be injected, evaluating the chemical-physical-mechanical compatibility with the masonry under repair. Cement-based mortars may damage the walls and especially the wall surfaces, due to salt production. The emergence of soluble salts in the mortar creates crusting on the wall surface, which is particularly damaging in the presence of ancient paintings, mosaics or frescoes. Such mortar should be used only after having accurately evaluated any eventual negative effects.

Interventions of repointing of mortar joints, when applied deeply on both sides of the walls, can improve the mechanical characteristics of the masonry, especially when the walls are not particularly thick. When used with medium or thick walls, in the cases where the external leaves are not well connected together, such an intervention is not sufficient to guarantee a consistent increase in resistance and it is therefore advisable to perform this intervention in combination with other strengthening techniques. Moreover, it is worth noting that this technique can cancel significant historic traces (original manufacturing of ancient masons) and, in case of exposed stone masonry, it can modify significantly the visual perception.

Quite recently some variations of the above-mentioned techniques have been proposed, all ascribable to the idea of reinforced repointing. In case of solid brick masonry, a thin bar (in stainless steel or composite materials) can be placed inside the joint, after scarification and before repointing. As far as irregular stone masonry is concerned, a thin flexible cable can be inserted along joints, without a regular meshing, before repointing (Reticulatus); some experimental tests proved the technique is able to provide some improvement of masonry properties.

DRAFT

The insertion of artificial transversal elements (diatones) is aimed at providing or improving the transversal connection between external leaves of a masonry wall, avoiding their separation due to instability phenomena under compression. Moreover, such interventions give the walls monolithic behaviour in cases of out-of-plane seismic actions. The intervention consists in coring transversally the wall (diameter around 100 mm), put a light reinforcement and inject cement or hydraulic mortar, in order to obtain a stiff element, which connect the external leaves by friction. Different technological details can be adopted (e.g. coring at smaller diameter and injecting cement-mortar in a proper sock, in order to prevent from diffusion in the masonry).

In cases where a small portion of masonry needs to be reinforced, because of local bulging, a valid solution is the use of anti-expulsion bars, made of thin transverse bars bolted with small flat washers to the external leaves of the wall. The scarce invasiveness attributed to this intervention renders it ideal in cases of wall leaves separation and, due to the possibility of post-tensioning the bar, a quote of deformation can be recovered. It is worth noting that bars must not be injected, in order to assure the maximum reversibility. This technique requires that masonry external leaf is not too irregular and of poor quality.

New techniques have been proposed quite recently, aimed at connecting the wall in the transverse direction and improving the monolithic behaviour. CAM system is a stitching method made by a regular mesh of thin steel bands, which pass the masonry through the thickness and tie block together. Ticorapsimo system is another stitching method made by thin cables, made of basalt composite fibres. It should be noted that these interventions may be effective in case of poor masonry but can be also quite invasive, even in relation to the extensions necessary for their effectiveness, and for these reasons they should be applied only when absolutely necessary.

The reinforced concrete jacketing of the masonry constitutes an invasive intervention and is not coherent with conservation principals: it consists in the application of steel meshes of the two sides of masonry wall, connected transverse bar and covered by a concrete layer of 4 to 6 cm (usually shotcrete is used). The strength improvement is obtained only if the technique is applied correctly (indeed, sometimes jacketing is applied only on one side or the transverse bars are omitted, because in stone masonry it is not simple to drill). From a seismic point of view, it is opportune to consider that it increases very much the stiffness of masonry panels; for this reason, the seismic behaviour of the building is strongly modified, not always in the positive direction (as the global displacement capacity is limited). Such technique can be used only in cases where the masonry is greatly damaged or incoherent and it is impossible to intervene with other methods; in any case, it is admissible not as a widespread intervention but only in limited portions of the masonry. In these cases, an alternative may also be the local demolition and reconstruction of that portion of masonry.

Jacketing with composite meshes (GFRP) is a recently proposed alternative, in which hydraulic mortar may be used (but usually cement concrete is preferred) and the thickness of the covering layers is smaller. The result is a lower increase of stiffness and masses, but the intervention still remains invasive and not advisable for a wide application in cultural heritage assets.

A very effective solution for the seismic strengthening of masonry panels is the application of the wall surfaces composite stripes (CFRP), glued with epoxy based resin. Stripes are usually disposed in two orthogonal directions; the adoption of a vertical and horizontal disposition (with horizontal stripes placed over the vertical ones, with the aim of acting like stirrups) is preferable in comparison with diagonal solutions.

DRAFT

The insertion inside masonry walls of post-tensioned vertical tie-rods is applicable only in specific cases and when the masonry has been proven to be able to support the increase in vertical load. In any case, the loss along time of the initial applied tension, caused by the long-term deformation of masonry, must be taken into consideration. Such a solution tends to modify the original behaviour of masonry constructions, in which masonry walls has no tensile strength and the stability is assured by the geometry; under this light, the method must be considered quite invasive and is advisable only in the absence of other alternatives.

In the case of decorated walls and frescoed surfaces, strengthening interventions cited herein can be utilised only with extreme caution, with the aid of experts who are specialised in the restoration of such surfaces. When possible, it is better to strengthen contiguous walls, with interventions of similar effectiveness, trying to minimize the drift demand in the decorated panels, by means of solutions that can be checked through proper models.

DRAFT

REFERENCES

Norms, Charts and Guidelines

Letter of the Ministry of Architecture and Construction of the Republic of Moldova no. 03-05/340 "Regarding the functioning of norms under construction on the territory of the Republic of Moldova", 1993. *Scrisoarea Ministerului Arhitecturii şi Construcţiilor al Republicii Moldova nr. 03-05/340 din 01.04.1993 "Referitor la funcţionarea normativelor în construcţie pe teritoriul Republicii Moldova"*.

СниП II-23-81*, 1981. “Стальные конструкции”.

CIB Publication 335, Guide for the Structural Rehabilitation of Heritage Buildings, 2010.

Circolare 2018. Istruzioni per l’applicazione dell’«Aggiornamento delle “Norme tecniche per le costruzioni”» di cui al decreto ministeriale 17 gennaio 2018. CIRCOLARE 21 gennaio 2019, n. 7 C.S.LL.PP, 2018.

FEMA 273. *NEHRP Guidelines for seismic rehabilitation of buildings*. Building Seismic Safety Council for the Federal Emergency Management Agency, USA, 1997.

FEMA 274. *NEHRP Commentary on the Guidelines for seismic rehabilitation of buildings*. Building Seismic Safety Council for the Federal Emergency Management Agency, USA, 1997.

FEMA 306. *Evaluation of earthquake damaged concrete and masonry wall buildings. Basic procedures manual*. Applied Technology Council for the Federal Emergency Management Agency, USA, 1998.

FEMA 307. *Evaluation of the earthquake-damaged concrete and masonry wall buildings – Technical resources.* Applied Technology Council for the Federal Emergency Management Agency, USA, 1998.

FEMA 356. *Prestandard and commentary for the seismic rehabilitation of buildings.* Applied Technology Council for the Federal Emergency Management Agency, USA, 2000.

Guidelines for the Assessment and Mitigation of Seismic Risk of Cultural Heritage with Reference to the 2008 Italian Building Code [Linee Guida per la Valutazione del Rischio Sismico del Patrimonio Culturale Allineate alle Nuove norme Tecniche per le Costruzioni]; G.U. No. 47; 2011. (In Italian)

ICOMOS Chart. *Principles for the Analysis Conservation and Structural Restoration of Architectural Heritage*. International scientific ommittee for analysis and restoration of structures of architectural heritage 2003.

ICOMOS Recommendations. *Recommendations for the analysis, conservation and structural restoration of architectural heritage*. International scientific ommittee for analysis and restoration of structures of architectural heritage, 2003.

ISO 13822. *Bases for design of structures — Assessment of existing structures*. 2010.

NTC 2008. *Norme Tecniche per le Costruzioni*. Decreto Ministero delle Infrastrutture, 14 gennaio 2008, G.U. n. 29 del 4 febbraio 2008 - Suppl. Ordinario n. 30 (In Italian).

NTC 2018. *Aggiornamento delle “Norme Tecniche per le Costruzioni”*. Decreto Ministero delle Infrastrutture e dei Trasporti, 17 gennaio 2018, Suppl. Ordinario n. 42 della G.U. del 20 febbraio 2018 - (In Italian).

ASCE/SEI 41/06. *Seismic Rehabilitation of Existing Buildings*. American Society of Civil Engineers, USA, 2006.

NCM F.03.02 – 2005, 2005. *“Proiectarea clădirilor cu pereţi din zidărie”.*

DRAFT

NCM F.02.02 – 2006 (2006). *"Calculul, proectarea şi alcătuirea elementelor de construcţii din beton armat şi beton precomprimat".*

Government Decision Nr. 936 from 16.08.2006 "Regulation on technical expertise in construction" (2006). *HOTĂRÎRE Nr. 936 din 16.08.2006 pentru aprobarea Regulamentului privind expertiza tehnică în construcţii* (http://lex.justice.md/index.php?action=view&view=doc&lang=1&id=317131).

EC8-1. *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings,* EN 1998-1. CEN, 1998.

EC8-3. *Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings,* EN 1998-3. CEN, 1998.

NCM A09.02-2005 "Technical maintenance, repair and reconstruction of housing, common and socio-cultural buildings" (2005). *NCM A.09.02 – 2005 "Deservirea tehnică, reparaţia şi reconstrucţia clădirilor de locuit, comunale şi social-culturale".*

Scientific papers

Abbas N., Calderini C., Cattari S., Lagomarsino S., Rossi M., Ginanni Corradini R., Marghella G., Piovanello V. (2010). Deliverable D4 – "Classification of the cultural heritage assets, description of the target performances and identification of damage measures", PERPETUATE Project.

Abruzzese D., Como M., Lanni G. (1992). On the lateral strength of multistory masonry walls with openings and horizontal reinforcing connections. Proc. X World Conference on Earthquake Engineering.

Acary V., Jean M. (1998). Numerical simulation of monuments by the contact dynamics method. Proc. of Monument 98 - Workshop on Seismic Performance of Monuments, Lisbon, 69-78.

Ali S.S., Page A.W. (1988). Finite element modelling for masonry subjected to concentrated loads. ASCE Journal of Structural Engineering, 114(8), 1761-1784.

Alpa G., Monetto I. (1994). Microstructural model for dry block masonry walls with in-plane loading, Journal of the mechanics and physics of solids, 42(7), 1159-1175.

Anthoine A. (1995). Derivation of in plane elastic characteristic of masonry through homogenization theory. International Journal of Solids and Structures, 32, 137-163.

Apostolou M., Gazetas G., Garinia E. (2007). Seismic response of slender rigid structures with foundation uplifting. Soil Dynamics and Earthquake Engineering, 27(7), 642-654.

Augusti G., Sinopoli A. (1992). Modelling the dynamics of large block structures, Meccanica, 27, 195-211.

Augusti, G., Ciampoli M. (1993). Protezione sismica degli oggetti d'arte: ulteriori studi sul moto di scivolamento con attrito. Proc. VI Convegno Nazionale ANIDIS, Perugia (in Italian).

Azevedo J., Sincraian G., Lemos J.V. (2000). Seismic behaviour of blocky masonry structures. Earthquake Spectra, 16(2), 337-365.

DRAFT

Baggio C., Trovalusci P. (1993). Discrete models for jointed block masonry walls. Proc. 6th NAMC, Philadelphia, 2, 939-949.

Baggio C., Trovalusci P. (1998). Limit Analysis for no-tension and frictional three-dimensional discrete systems. Mechanics of Structures and Machines, 26(3), 287-304.

Baggio C., Trovalusci P. (2000). Collapse behaviour of three-dimensional brick-block systems using non-linear programming. Structural Engineering and Mechanics, 10(2), 181-195.

Belmouden Y., Lestuzzi P. (2009). An equivalent frame model for seismic analysis of masonry and reinforced concrete buildings. Construction and Building Material, 23(1), 40-53.

Benedetti D., Carydis P., Pezzoli P. (1998). Shaking table tests on 24 simple masonry buildings. Earthquake Engineering and Structural Dynamics, 27, 67-90.

Berto L., Saetta A., Scotta R., Vitaliani R. (2002). An orthotropic damage model for masonry structures. International Journal for Numerical Methods in Engineering, 55, 127-157.

Bicanic N., Ponniah D., Robinson J. (2001). Discontinuous deformation analysis of masonry bridges. Computational Modelling of Masonry, Brickwork and Blockwork Structures, 177-196.

Bosiljkov V., Bokan-Bosiljkov V., Žarnic R. (2007). Numerical models for seismic assessment of historic structures. International Journal of Engineering Modelling, 20 (10).

Braga F., Dolce M. (1982). Un metodo per l'analisi di edifici multipiano in muratura antisismici, Proc. 6th I.B.Ma.C., Roma (in Italian).

Braga F., Liberatore D. (1990). A finite element for the analysis of the response of masonry buildings. Proc. of the 5th North American Masonry Conference, Urbana, 201-212.

Brasile S., Casciaro R., Formica G. (2007). Multilevel approach for brick masonry walls - Part II: On the use of equivalent continua, Computer Methods in Applied Mechanics and Engineering, 196, 4801-4810.

Brasile S., Casciaro R., Formica G. (2007). Multilevel approach for brick masonry walls part I: A numerical strategy for the nonlinear analysis. Computer Methods in Applied Mechanics and Engineering 196, 4934-4951.

Brencich A., Gambarotta L., Lagomarsino S. (1998). A macroelement approach to the three-dimensional seismic analysis of masonry buildings, Proc. XI European Conference on Earthquake Engineering, Paris, A.A. Balkema (Abstract Volume & CD-ROM).

Briccoli Bati S., Paradiso M., Tempesta G. (1999). Analysis of voussoir arches modelled on rigid blocks and elastic unilateral contact constraints. Proc. Assisi ’99-Workshop on Seismic Performance of Built Heritage in Small Historical Centres, CICOP Italia, 63-71.

Briccoli Bati, S., Paradiso M., Tempesta G. (1995). A numerical procedure for the analysis of brickwork walls. Proc. III International Symposium on Computer Methods in Structural Masonry, Lisbon, 303-312.

Brignola A. (2009). Evaluation of the in-plane stiffness of timber floors for the performance-based retrofit of URM buildings." PhD Thesis, University of Genoa.

DRAFT

Brignola A., Pampanin S., Podestà S. (2009). Evaluation and control of the in-plane stiffness of timber floors for the performance-based retrofit of URM buildings. Bulletin of the New Zealand Society for Earthquake Engineering.

Calderini C. Lagomarsino S. (2006). A micromechanical inelastic model for historical masonry. Journal Earthquake Engineering, 10(4), 453-479.

Calderini C., Cattari S., Lagomarsino S. (2009). In-plane strength of unreinforced masonry piers, Earthquake Engineering and Structural Dynamics, 38(2), 243-267.

Calderini C., Lagomarsino S. (2008). A continuum model for in-plane anisotropic inelastic behaviour of masonry. ASCE Journal of Structural Engineering, 134(2), 209-220.

Calderoni B., Cordasco E.A., Lenza P. (2007). Analisi teorico sperimentale del comportamento della fascia di piano delle pareti murarie per azioni sismiche, Proc. XII Congresso Nazionale ANIDIS, Pisa (in Italian).

Calderoni B., Cordasco E.A., Lenza P. (2007). Il ruolo della fascia di piano nel comportamento sismico degli edifici in muratura. Ingegneria Sismica, 1, 26-40 (in Italian).

Calderoni B., Lenza P., Pagano M. (1989). Attuali prospettive per l'analisi sismica non lineare di edifici in muratura". Proc. IV Congresso Nazionale ANIDIS, Milano (in Italian).

Calderoni B., Marone P., Pagano M. (1987). Modelli per la verifica statica degli edifici in muratura in zona sismica. Ingegneria sismica, 3, 19-27 (in Italian).

Caliò I., Cannizzaro F., Marletta M. (2010). A discrete element for modelling masonry vaults. Proc. VII International Conference on Structural Analysis of Historical Construction, Shanghai, 1, 447-452.

Caliò I., Marletta M., Pantò B. (2007). Un macroelemento in grado di cogliere il comportamento nel piano e fuori piano di pareti murarie. Proc. XII Congresso Nazionale ANIDIS, Pisa (in Italian).

Capecchi D., Giannini R., Masiani R. (1996). Motion of a rigid body with a rounded base due to harmonic excitation. International Journal of Non-Linear Mechanics, 31(2), 175-191.

Carocci C.F. (2001). Guidelines for the safety and preservation of historical centres in seismic area, Proc. III International Seminar on Structural Analysis of Historical Constructions, Guimarães (Portugal), 145-165.

Casapulla C., D'Ayala D. (2001). Lower bound approach to the limit analysis of 3D vaulted block masonry structures. Proc. V International Symposium on Computer Methods in Structural Masonry, Rome, 28-36.

Casapulla C., De Riggi T., Jossa P. (1998). Il ruolo dell'attrito nella resistenza di pareti murarie. Ingegneria Sismica, 3, 32-40.

Casapulla C., D'Ayala D.F. (2006). In plane collapse behaviour of masonry walls with frictional resistance and openings. Proc. V Conference on Structural Analysis of Historical Construction, New Delhi.

Casolo S. (2006). Macroscopic modelling of structured materials: Relationship between orthotropic cosserat continuum and rigid elements. International Journal of Solids and Structures, 43, 475-496.

Casolo S., Peña F. (2007). Rigid element model for in-plane dynamics of masonry walls considering hysteretic behaviour and damage. Earthquake Engineering and Structural Dynamics, 36(8), 1029-1048.

DRAFT

Cattari S. (2007). Modellazione a telaio equivalente di strutture esistenti in muratura e miste muratura-c.a.: formulazione di modelli sintetici, PhD Thesis, University of Genoa (in Italian).

Cattari S., Lagomarsino S. (2008). "A strength criterion for the flexural behaviour of spandrels in un-reinforced masonry walls", Proceedings of 14th World Conference on Earthquake Engineering, Beijing, China, CD-Rom.

Cattari S., Lagomarsino S. (2008). A strength criterion for the flexural behaviour of spandrel in un-reinforced masonry walls. Proc. 14th World Conference on Earthquake Engineering, Beijng, Cina.

Cattari S., Lagomarsino S. (2009). Modelling the seismic response of unreinforced existing masonry buildings: a critical review of some models proposed by codes, Proc. 11th Canadian Masonry Symposium, Toronto, Ontario, CD-Rom.

Cattari S., Lagomarsino S., Resemini S. (2008). Modelling of vaults as equivalent diaphragms in 3D seismic analysis of masonry buildings, Proceedings of 6th International Conference on Structural Analysis of Historic Construction, Bath, D'Ayala & Fodde eds, Taylor & Francis Group.

Cecchi A., Milani G. (2008). A kinematic limit analysis model for thick English bond masonry walls. International Journal of Solids and Structures, 45, 1302-1311.

Cecchi A., Milani G., Tralli A. (2005). Validation of analytical multiparameter homogenisation models for out-of-plane loaded masonry walls by means of finite element method. ASCE Journal of Engineering Mechanics, 131(2), 185-198.

Cecchi A., Milani G., Tralli A. (2007). A Reissner-Mindlin limit analysis model for out-of-plane loaded running bond masonry walls. International Journal of Solids and Structures, 44, 1438-1460.

Cecchi A., Sab K. (2002a). A multi-parameter homogenization study for modelling elastic masonry. European Journal of Mechanics A/Solids, 21, 249-268.

Cecchi A., Sab K. (2002b). Out of plane model for heterogeneous periodic materials: the case of masonry. European Journal of Mechanics A/Solids, 21, 715-746.

Cecchi A., Sab K. (2008). Discrete and continuous models for in plane loaded random elastic brickwork. European Journal of Mechanics A/Solids, 28(3), 610- 625.

Cerrolaza M., Sulem J., Elbied A. (1999). A Cosserat non-linear finite element analysis software for blocky structures. Advances in Engineering Software, 30, 69-83.

Chen S.Y., Moon F.L., Yi T. (2008). A macroelement for the nonlinear analysis of in-plane unreinforced masonry piers. Engineering Structures, 30, 2242-2252.

Clemente P. (1998). Introduction to dynamics of stone arches. Earthquake Engineering and Structural Dynamics, 27, 513-522.

Clemente P., Raithel A. (1998). The mechanism model in the seismic check of stone arches. Proc. Arch Bridges, Sinopoli A (ed.). Balkema, Rotterdam, 123-129.

Cluni F., Gusella V. (2004). Homogenization of non-periodic masonry structures. International Journal of Solids and Structures, 41 (7), 1911-1923.

DRAFT

Como M., Grimaldi A. (1985). Application of the masonry unilateral model to the limit analysis of buildings loaded by horizontal forces. Proc. 2nd Meeting on 'Unilateral problems in structural analysis', Prescudin, 82-95.

Como M., Grimaldi A., Lanni G. (1988). New results on the horizontal strength evaluation of masonry buildings and monuments. Proc. IX World Conference on Earthquake Engineering, Tokyo-Kyoto.

Corigliano A., Maier G. (1995). Dynamic shakedown analysis and bounds for elasto-plastic structures with non associative, internal variable constitutive laws. International of Journal of Solids and Structures, 32 (21), 3145-3166.

Cundall P.A. (1976). Explicit finite difference methods in geomechanics. Numerical Methods in Engineering, Blacksburg, Virginia, 1, 132-150.

Cuomo M., Ventura G. (2000). A complementary energy formulation of no tension masonry-like solids. Computer Methods in Applied Mechanics and Engineering, 189, 313-339.

D'Asdia P., Viskovic A. (1994). L'analisi sismica degli edifici in muratura. Ingegneria Sismica, XI(1), 32-42 (in Italian).

D'Ayala D., Speranza E. (2003). Definition of collapse mechanisms and seismic vulnerability of historic masonry buildings. Earthquake Spectra, 19, 479-509.

D'Ayala D., Tomasoni E. (2008). Structural behaviour of masonry vaults: limit state analysis with finite friction. structural analysis of historic construction: preserving safety and significance. Proc. VI International Conference on Structural Analysis of Historic Construction, Bath (UK), 47-61.

Dazio A., Beyer K. (2010). Seismic behaviour of different types of masonry spandrels, Proc. XIV European Conference on Earthquake Engineering, Ohrid, CD-Rom.

De Borst R., Feenstra P.H., Pamin J., Sluys L.G. (1994). Some current issues in computational mechanics of concrete structures. In: Mang N.B., Bicanic N., De Borst R. (editors). Computational Modelling of Concrete Structures, vol. 1. Swansea (UK): Pinerdge Press, 283–302.

De Borst R., Sluys L.G., Mulhaus H.B., Pamin J. (1993). Fundamental issues in finite element analyses of localization of deformation. Engineering Computations, 10, 99-121.

De Buhan P., De Felice G. (1997). A homogenisation approach to the ultimate strength of brick masonry. Journal of the Mechanics and Physics of Solids, 45 (7), 1085-1104.

De Felice G. (2001). Overall elastic properties of brickwork via homogenisation. Proc. of the Int. Conf. on Structural Engineering Mechanics and Computation, Vol. I, 411-419.

De Felice G. (2010). Out-of-plane seismic capacity of masonry depending on wall section morphology. International Journal of Architectural Heritage, in press.

De Felice G., Giannini R. (2001). Out-of-plane seismic resistance of masonry walls. Journal of Earthquake Engineering, 5(2), 253-271.

De Lorenzis L., DeJong M., Ochsendorf J. (2007). Failure of masonry arches under impulse base motion. Earthquake Engineering Structural Dynamics. 36, 2119-2136.

DRAFT

De Luca A., Giordano A., Mele E. (2004). A simplified procedure for assessing the seismic capacity of masonry arches. Engineering Structures, 26, 1915-1929.

Del Piero G. (1989). Constitutive equation and compatibility of the external loads for linear elastic masonry-like materials. Meccanica, 24, 150-162.

Di Pasquale S. (1992). New trends in the analysis of masonry structures. Meccanica, 27, 173-184.

Doherty K., Griffith M.C., Lam N., Wilson J. (2002). Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls. Earthquake Engineering and Structural Dynamics, 31(4), 833-850.

Dvorak G. (1992). Transformation field analysis of inelastic composite materials. Proc. Royal Society of London A 437, 311-327.

Ferris M., Tin-Loi F. (2001). Limit analysis of frictional block assemblies as a mathematical program with complementarity constraints, International of Journal and Mechanical Science, 43, 209-224.

Ferris M., Tin-Loi F. (2001). Limit analysis of frictional block assemblies as a mathematical program with complementarity constraints. International Journal of Mechanical Sciences, 43, 209-224.

Feyel F., Chaboche J.L. (2000). FEM multiscale approach for modelling the elastoviscoplastic behavior of long fibre SiC/Ti composite materials. Computer Methods in Applied Mechanics and Engineering, 183, 309-330.

Fish J., Shek K. (2000). Multiscale analysis of composite materials and structures. Composites Science and Technology, 60, 2547-2556.

Fish J., Wagiman A. (1993). Multiscale finite element method for a locally nonperiodic heterogeneous medium. Computational Mechanics 12, 64-81.

Fusier F., Vignoli A. (1993). Proposta di un metodo di calcolo per edifici in muratura sottoposti ad azioni orizzontali. Ingegneria Sismica, Anno X, N.1 (in Italian).

Galasco A., Lagomarsino S., Penna A. (2006). On The Use Of Pushover Analysis For Existing Masonry Buildings, Proc. First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland, 1080, CD-ROM.

Galasco A., Lagomarsino S., Penna A., Cattari S. (2009). TREMURI program: Seismic Analyses of 3D Masonry Buildings, University of Genoa (mailto: tremuri@gmail.com).

Galasco A., Lagomarsino S., Penna A., Resemini S. (2004). Non-linear Seismic Analysis of Masonry Structures. Proc. XIII World Conference on Earthquake Engineering, Vancouver, Paper 843.

Gambarotta L., Lagomarsino S. (1996). Sulla risposta dinamica di pareti in muratura. Proc. Convegno Nazionale "La Meccanica delle Murature tra Teoria e Progetto", Messina, Pitagora Ed. Bologna (in Italian).

Gambarotta L., Lagomarsino S. (1997). Damage models for the seismic response of brick masonry shear walls. Part I: The mortar joint model and its applications; Part II: the continuum model and its applications. Earthquake Engineering Structural and Dynamics, 26, 423-462.

Gattesco N., Clemente I., Macorini L., Noè S. (2008). Experimental investigation on the behaviour of spandrels in ancient masonry buildings. Proceedings of the 14th WCEE, Beijng, Cina, CD-ROM.

DRAFT

Giambanco G., Rizzo S., Spallino R. (2001). Numerical analysis of masonry structures via interface models. Computer Methods in Applied Mechanics and Engineering, 190, 6493-6511.

Giannini, R. (1984). Analisi dinamica di un sistema di blocchi sovrapposti. Proc. II Convegno Nazionale ANIDIS, Rapallo, 647-662 (in Italian).

Giaquinta M., Giusti E. (1985). Researches on the equilibrium of masonry structures. Archive for rational mechanics and analysis, 88, 359-392.

Gilbert M., Casapulla C., Ahmed H.M. (2006). Limit analysis of masonry block structures with non-associative frictional joints using linear programming. Computer & Structures, 84, 873-887.

Gilbert M., Melbourne C. (1994). Rigid block analysis of masonry structures. The Structural Engineering, 72(21), 356-361.

Giordano A., De Luca A., Mele E., Romano A. (2004). A simple formula for predicting the horizontal capacity of masonry portal frames. Engineering Structures, 29, 2109–2123.

Giuffré A. (1989). Mechanics of historical masonry and strengthening criteria, XV Regional Seminar on Earthquake engineering, Ravello, Italy, Edizioni Kappa: Roma.

Giuffre' A. (1991). Letture sulla meccanica delle murature antiche, Edizioni Kappa: Roma.

Giuffré A., Carocci C.F. (1993). Statica e dinamica delle costruzioni muraraie storiche. Proc. Of the Conference “Tufo calcareo e la pietra leccese”, CNR, Bari.

Graziotti F., Magenes G., Penna A. (2009). Progetto di una sperimentazione su elementi di fascia muraria. Rapporto Reluis, Allegato 4.3-UR01-1, University of Pavia and EUCENTRE, Pavia, Italy (in Italian).

Graziotti F., Magenes G., Penna A. (2009). Progetto di una sperimentazione su elementi di fascia muraria. Rapporto Reluis, Allegato 4.3-UR01-1, Università di Pavia e EUCENTRE, Pavia, Italy.

HAZUS-MH MR1. *Multi-Hazard Loss Estimation Methodology: Earthquake Model*, Department of Homeland Security, Federal Emergency Management Agency, USA, 2003.

Heyman J. (1966). The stone skeleton. International Journal of Solids and Structures, 2, 249-279.

Heyman J. (1982). The Masonry Arch, Ellis Horwood-Wiley: West Sussex, U.K.

Heyman, J. (1995). The stone skeleton. Structural engineering of masonry architecture, Cambridge University Press: New York.

Housner G.W. (1963). The behavior of inverted pendulum structures during earthquakes. Bulletin of the Seismological Society of America. 53(2), 403-417.

Hughes T.J.R. (1995). Multiscale phenomena: Green's functions, the Dirichelet-to-Neumann formulation, subgrid scale models, bubbles, and the origins of stabilized methods. Computer Methods in Applied Mechanics and Engineering, 127, 387-401.

Ishiyama Y. (1982). Motion of rigid bodies and criteria for overturning by earthquake excitations. Earthquake Engineering and Structural Dynamics, 10, 635-650.

DRAFT

Ishiyama Y. (1982). Motion of rigid bodies and criteria for overturning by earthquake excitations. Earthquake Engineering and Structural Dynamics, 10, 635-650.

Jean M. (1995). Frictional contact in collections of rigid or deformable bodies: numerical simulation of geomaterial motions. Mechanics of Geomaterial Interfaces, Selvadurai A.P.S. and Boulon M.J. (edt.), Amsterdam: Elzevier, 463-486.

Jeong M.J., Suzuki K., Yim C.S. (2003). Chaotic rocking behavior of freestanding objects with sliding motion. Journal of sound and vibration, 262, 1091-1112.

Kozlov S.M. (1980). Averaging of random operators. Math. USSR Sb. 37 (2), 167-180.

Lachihab A., Sab K. (2005). Aggregate composites: A contact based modelling. Computational Materials Science, 33, 467-490.

Lagomarsino S., Cattari S. (2009). Non linear seismic analysis of masonry buildings by the equivalent frame model, Proceedings of 11th D-A-CH Conference, Zurich.

Lee J.S., Pande G.N., Middleton J., Kralj B. (1996). Numerical modelling of brick masonry panels subject to lateral loadings, Computer & Structures, 61(4), 735-745.

Lemos J.V. (2007). Discrete element modelling of masonry structures. International Journal of Architectural Heritage, 1(2), 190-213.

Lipscombe P.R., Pellegrino S. (1993). Free rocking of prismatic blocks. Journal of Engineering Mechanics, 119 (7), 1387-1410.

Livesley R.K. (1978). Limit analysis of structures formed for rigid blocks. International Journal of Numerical Method in Engineering, 12, 1853-1871.

Lofti H.R., Shing P.B. (1997). Multisurface interface model for analysis of masonry structures. Journal of Engineering Mechanics, ASCE, 123(7), 660-668.

Lotfi H.R., Shing P.B. (1991). An appraisal of smeared crack models for masonry shear wall analysis. Computer and Structures, 41(3), 413-425.

Lourenço P.B. (2000). Anisotropic softening model for masonry plates and shells. ASCE Journal of structural engineering, 126(9), 1008-1015.

Lourenço P.B., Borst R. Rots, J.G. (1997). A plane stress softening plasticity model for orthotropic materials. International Journal for Numerical Methods in Engineering, 40, 4033-4057.

Lourenço P.B., Rots J.G. (1997). Multisurface interface model for analysis of masonry structures. ASCE Journal of Engineering Mechanics, 123(7), 660-668.

Lourenço P.B., Rots J.G. (1997). On the use of homogenisation techniques for the analysis of masonry structures. Masonry International, 11(1), 26-32.

Lourenço P.B., Rots J.G., Blaauwendraad J. (1998). Continuum model for masonry: parameter estimation and validation. ASCE Journal of Structural Engineering, 124(6), 642-652.

DRAFT

Luciano R., Sacco E. (1997). Homogenization technique and damage model for old masonry material. International Journal of Solids and Structures, 34, 3191-3208.

Ma M.A., Pan A.D.E., Luan M., Gebara J.M. (1996). Seismic analysis of stone arch bridges using discontinuous deformation analysis. Proc. XI World Conference on Earthquake Engineering, Amsterdam, Paper no.1551.

Magenes G. (2000). A method for pushover analysis in seismic assessment of masonry buildings. Proc. XII World Conference on Earthquake Engineering, Auckland.

Magenes G. (2007). Masonry building design in seismic areas: recent experiences and prospects from a European standpoint. In Proceedings of 1st ECEES, Geneva, Switzerland, CD-Rom.

Magenes G., Braggio C., Bolognini D. (2001). Metodi semplificati per l'analisi sismica non lineare di edifici in muratura, CNR-GNDT, Roma (in Italian).

Magenes G., Della Fontana A. (1998). Simplified non-linear seismic analysis of masonry buildings. Proc. of the British Masonry Society, 8, 190-195.

Magenes G., Della Fontana A. (1998). Simplified non-linear seismic analysis of masonry buildings. Proc. of the British Masonry Society, 8, 190-195.

Maier G., Nappi A., Papa E. (1991). Damage model for masonry as a composite material: a numerical and experimental analysis. Proc. of the 3rd International Conference on 'Constitutive laws for engineering materials: theory and application', Tucson, USA, 427-432.

Mamaghani I.H.P., Aydan O., Kajikawa Y. (1999). Analysis of masonry structures under static and dynamic loading by discrete finite element method. Journal of Structural Mechanics and Earthquake Engineering, 16(2), 75-86.

Masiani R., Rizzi N., Trovalusci P. (1995). Masonry as structured continuum. Meccanica, 30, 673-683.

Massart T.J., Peerlings R.H.J., Geers M.G.D. (2004). Mesoscopic modelling of failure and damage-induced anisotropy in brick masonry. European Journal of Mechanics A/Solids, 23, 719-735.

Milani G., Beyer K., Dazio A. (2009). Upper bound limit analysis of meso-mechanical spandrel models for the pushover analysis of 2D masonry frames. Engineering Structures, 31(11), 2696-2710.

Milani G., Lourenço P.B., Tralli A. (2006). Homogenised limit analysis of masonry walls, Part I: Failure surfaces, Part II: Structural examples. Computers and Structures, 84, 166-195.

Mistler M., Anthoine A., Butenweg C. (2007). In-plane and out-of-plane homogenization of masonry. Computers & Structures, 85, 1321-1330.

Moon F.L., Tianyi Y., Leon R.T., Kahn L.F. (2006). Recommendations for Seismic Evaluation and Retrofit of Low-Rise URM Structures, Journal of Structural Engineering, ASCE, 132(5).

Munjiza A., Owen D.R.J., Bicanic N. (1995). A combined finite-discrete element method in transient dynamics of fracturing solids. Engineering Computations, 12, 145-174.

Nadler B., Papadopoulos P., Steigmann D.J. (2006). Multiscale constitutive modelling and numerical simulation of fabric material. International Journal of Solids and Structures, 43, 206-221.

DRAFT

Nozaki K., Terumichi Y., Nishimura K., Sogabe K. (2009). Study of rocking motion of rigid body with slide contact. Journal of Mechanical Science and Technology, 23, 1001-1007.

Ochsendorf J. (2002). Collapse of Masonry Structures. PhD thesis, University of Cambridge.

Ochsendorf J., Hernando J.I., Huerta S. (2004). Collapse of Masonry Buttresses. Journal of Architectural Engineering, 10(3), 88-97.

Oppenheim I.J. (1992). The masonry arch as a four-link mechanism under base motion. Earthquake Engineering and Structural Dynamics, 21, 1005-1017.

Orduna A. (2003). Seismic assessment of ancient masonry structures by rigid blocks limit analysis. PhD thesis, University of Minho, Guimaraes.

Orduna A., Lourenço P.B. (2005). Three-dimensional limit analysis of rigid blocks assemblages. Part I: Torsion failure on frictional interfaces and limit analysis formulation; Part II: load-path following solution procedure and validation. International Journal of Solids and Structure, 42(18-19), 5140-5180.

Orduna A., Roeder G., Araiza J.C. (2006). Development of macro-block models for masonry walls subject to lateral loading. Proc. V International Conference on Structural Analysis of Historical Constructions, New Delhi, 1075-1080.

Page A.W. (1978). Finite element model for masonry. Journal of Structural Division, ASCE, 104 (ST8), 1267-1285.

Pagnoni T. (1994). Seismic analysis of masonry and block structures with the discrete element method. Proc. X European Conference on Earthquake Engineering, 3, 1669-1674.

Pagnoni T., Vanzi I. (1995). Experimental and numerical study of the seismic response of block structures. Computer Methods in Structural Masonry, 3, 213-222.

Pande G.N., Liang J.X., Middleton J. (1989). Equivalent elastic moduli for brick masonry. Computers an Geotechnics, 8, 243-265.

Panzeca T., Polizzotto C. (1988). Constitutive equations for no-tension materials. Meccanica, 23, 88-93.

Papa E., Nappi, A. (1997). Numerical modelling of masonry: a material model accounting for damage effects and plastic strains. Applied Mathematical Modelling, 21, 319-335.

Papanicolaou G.C., Varadhan S.R.S. (1979). Boundary value problems with rapidly oscillating random coefficients. In: Colloquia Mathematica Societatis Janos Bolyai, vol. 2. North-Holland, Elsevier Science Publishers: Amsterdam, New York, Oxford, 835-873.

Papantonopoulos C., Psycharis N., Papastamatiou D., Lemos J., Mouzakis H. (2002). Numerical prediction of the earthquake response of classical columns using the distinct element method. Earthquake Engineering and Structural Dynamics, 31, 1699-1717.

Park Y., Ang A. (1985). Mechanistic seismic damage model for reinforced concrete. Journal of Structural Engineering, 111, 722-739.

DRAFT

Pegon P., Anthoine A. (1997). Numerical Strategies for Solving Continuum Damage Problems with Softening: Application to the Homogenization of Masonry. Computer and Structures, 64, 623-642.

Penelis G.G. (2006). An Efficient Approach for Pushover Analysis of Unreinforced Masonry (URM) Structures. Journal of Earthquake Engineering, 10(3), 359-379.

Penna A. (2002). A macro-element procedure for the dynamic non-linear analysis of masonry buildings. PhD Thesis, Politecnico di Milano.

Penna A., Cattari S., Galasco A., Lagomarsino S. (2004). Seismic Assessment of Masonry Structures by Non-linear Macro-element Analysis. Proc. IV International Conference on Structural Analysis of Historical Construction, Padova, 2, 1157-1164.

Pietruszczak S., Niu X. (1991). A mathematical description of macroscopic behaviour of brick masonry. International Journal Solids Structures, 29(5), 531-546.

Pietruszczak S., Ushaksaraei R. (2003). Description of inelastic behaviour of structural masonry. International Journal of Solids and Structures, 40, 4003-4019.
Pleșca, V., Fagurel, R. (2017). Instituirea urmăririi speciale asupra centrului istoric al Chișinăului. BULLETIN INCERCOM, 69-102.

Priestley N. (1985). Seismic Behaviour of Unreinforced Masonry Walls. Bulletin NZNSEE, 18(2), 191-205.

Prieto F., Lourenço P.B. (2005). On the rocking behaviour of rigid object, Meccanica, 40, 121-133.

Psycharis I.N., Lemos J.V., Papastamatiou D.Y., Zambas C., Papantonopoulos C. (2003) Numerical study of the seismic behaviour of a part of the Parthenon Pronaos. Earthquake Engineering & Structural Dynamics, 32 , 2063-2084.

Roberti G.M., Spina O. (2001). Discrete element analysis of the Sardinian 'Nuraghe'. Historical Constructions, (eds., P.B. Lourenco and P. Roca). Guimaraes: Universidade do Minho, 719-727.

Roca P., Molins C., Marì A.R. (2005). Strength Capacity of Masonry Wall Structures by the Equivalent Frame Method. ASCE Journal of Structural Engineering, 131(10), 1601-1610.

Romano G., Romano M. (1979). Sulla soluzione di problemi strutturali in presenza di legami costitutivi unilaterali. Rend. Accademia Nazionale dei Lincei, Classe di Scienze Fisiche, Matematiche e Naturali, Serie VIII, vol. LXVII (in Italian).

Romano G., Sacco E. (1984). Sul calcolo di strutture non resistenti a trazione. Proc. of VII AIMETA National Congress, Trieste, Italy, V, 217-226 (in Italian).

Rots J.G. (1991). Numerical simulation of cracking in structural masonry. Heron, 36(2), 49-63.

Sab K. (1992). On the homogenization and the simulation of random materials. European Journal and Mechanics A/Solids 11 (5), 585-607.

Sacco E. (1990). Modellazione e calcolo di strutture in materiale non reagente a trazione, Rend. Mat. Acc. Lincei, Serie IX, vol. I, 235-258 (in Italian).

Sacco E. (2009). A nonlinear homogenization procedure for periodic masonry. European Journal of Mechanics. A/Solids, 28, 209-222.

DRAFT

Shi G.H., Goodman R.E. (1988). Discontinuous deformation analysis – A new method for computing stress, strain and sliding of block systems. Key Questions in Rock Mechanics, Cundall P.A., Sterling R., Starfield A. (edt.), Rotterdam: Balkema, 181-186.

Sinopoli A. (1989). Dynamic analysis of a stone column excited by a sine wave ground motion. Applied Mechanics Reviews, 44(11), 246-255.

Sinopoli A., Sepe, V. (1993). Coupled motion in the dynamic analysis of a three-block structure. Applied Mechanics Reviews, 46 (10), 185-197.

Sorrentino L., Masiani R. (2007). Risposta fuori dal piano di pareti murarie, libere e vincolate in sommità a segnali naturali, Atti del XII Convegno "L'Ingegneria Sismica in Italia" (in Italian).

Sorrentino L., Masiani R., Griffith M.C. (2008). The vertical spanning strip wall as a coupled rocking rigid body assembly. Structural Engineering Mechanics, 29(4), 433-453.

Spanos D., Roussis P.C., Politis N. (2001). Dynamic analysis of stacked rigid blocks. Soil Dynamics and Earthquake Engineering, 21, 559-578.

Spence R., Coburn A. (1992). Strengthening building of stone masonry to resist earthquakes. Meccanica, 27, 213-221.

Sutcliffe D.J., Yu H.S., Page A.W. (2001). Lower bound limit analysis of unreinforced masonry shear walls. Computers and Structures, 79, 1295-1312.

Tomaževic M. (1978). The computer program POR. Report ZRMK.

Tomaževic M., Weiss P. (1990). A rational experimentally based method for the verification of earthquake resistance of masonry buildings. Proc. IV U.S. National Conference on Earthquake Engineering, Palm Springs, Vol. 2, 349-359.

Trovalusci P., Masiani R. (2003). Non-linear micropolar and classical continua for anisotropic discontinuous materials, 40, 1281-1297.

Acknowledgements

The authors thanks Evgeni Cutia from the Technical University of Moldova for his support in understanding Moldovan normative framework and norms.

DRAFT